(2)

US Army Corps
of Engineers
Los Angeles District

SANTA ANA RIVER BASIN, CALIFORNIA

AD-A204 540

Santa Ana River

Design Memorandum No. 1

PHASE II GDM ON THE SANTA ANA RIVER MAINSTEM including Santiago Creek

DTIC
ELECTE
2 FEB 1989
S D
E

This document has been approved for public release and sale; its distribution is unlimited.

VOLUME 1
SEVEN OAKS DAM
INCLUDING APPENDIX A

89 2 2 008

August 1988

REPORT DOCUMENTATION PAGE

READ INSTRUCTIONS BEFORE COMPLETING FORM

1. REPORT NUMBER Design Memorandum No. 1	2. GOVT ACCESSION NO.	3. RECIPIENT'S CATALOG NUMBER
4. TITLE (and Subtitle) Phase II GDM on the Santa Ana River Mainstem Including Santiago Creek Volume 1, Seven Oaks Dam, Including Appendix A		5. TYPE OF REPORT & PERIOD COVERED Final 6. PERFORMING ORG. REPORT NUMBER
7. AUTHOR(s) U S Army Corps of Engineers Los Angeles District		8. CONTRACT OR GRANT NUMBER(s)
9. PERFORMING ORGANIZATION NAME AND ADDRESS Engineering Division 300 N Los Angeles Street Los Angeles, CA 90012		10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS
11. CONTROLLING OFFICE NAME AND ADDRESS Project Management Brach 300 N Los Angeles Street Los Angeles, CA 90012		12. REPORT DATE August 1988 13. NUMBER OF PAGES 822
14. MONITORING AGENCY NAME & ADDRESS (if different from Controlling Office) Same As Controlling Office		15. SECURITY CLASS. (of this report) Unclasssified 15a. DECLASSIFICATION/DOWNGRADING SCHEDULE

16. DISTRIBUTION STATEMENT (of this Report)

Approved for Public Release; Distribution Unlimited

17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)

Approved for Public Release; Distribution Unlimited

18. SUPPLEMENTARY NOTES

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

- Dam
- Spillway
- Outlet Works
- Reservoir
- Probable Maximun Flood
- Project Plan
- Hydraulic Design
- Structural Design
- Floodway Delineation

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

This volume accompanies the Main Report and Supplemental Environmental Impact Statement for the Phase II General Design Memorandum for the Santa Ana River Mainstem including Santiago Creek and contains the general design for the Seven Oaks Dam and the preliminary floodplain and floodway delineation between Seven Oaks Dam and Prado Reservoir. The project economic data is presented in Volume 9, "Economics and Public Comments and Response."

DD FORM 1 JAN 73 1473 EDITION OF 1 NOV 65 IS OBSOLETE

Unclassified

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

Design Memorandum No. 1
Volume 1
Santa Ana River Mainstem
Including Santiago Creek, California
Phase II General Design Memorandum

SEVEN OAKS DAM

Accession For	
NTIS GRA&I	☒
DTIC TAB	☐
Unannounced	☐
Justification	
By	
Distribution/	
Availability Codes	
Dist	Avail and/or Special
A-1	



SYLLABUS

This volume accompanies the Main Report and Supplemental Environmental Impact Statement for the Phase II General Design Memorandum for the Santa Ana River Mainstem including Santiago Creek and contains the general design for the Seven Oaks Dam and the preliminary floodplain and floodway delineation between Seven Oaks Dam and Prado Reservoir. The project economic data is presented in Volume 9, "Economics and Public Comment and Response."

Seven Oaks Dam, located 8 miles northeast of the city of Redlands in San Bernardino County, will be an earth and rock fill structure with a height of 550 feet above the existing streambed, crest width of 40 feet, crest length of 2,980 feet and crest elevation at 2,610 feet NGVD. The upstream and downstream slopes will be 1 vertical on 2 horizontal. The dam will have a storage capacity of 145,600 acre-feet at the spillway crest elevation of 2,580 feet NGVD.

A detached spillway will be located about 1,700 feet east of the dam. It will have a trapezoidal cross-section with a base width of 500 feet and sideslopes averaging 1 vertical on 1 horizontal. The spillway will be about 1,400 feet long and unlined except for a concrete control sill at the crest, approximately 1,000 feet from the downstream end.

The outlet works tunnel will be located within the east abutment. The tunnel, excavated in rock and lined with reinforced concrete, will be approximately 1,750 feet long. Total first cost of the project is estimated at $316,283,000.

PHASE II GDM LISTING OF VOLUMES

Main Report and Supplemental Environmental Impact Statement

Volume 1	Seven Oaks Dam
Volume 2	Prado Dam
Volume 3	Lower Santa Ana River (Prado Dam to Pacific Ocean)
Volume 4	Mill Creek Levee
Volume 5	Oak Street Drain
Volume 6	Santiago Creek
Volume 7	Hydrology
Volume 8	Environmental
Volume 9	Economics and Public Comment and Response

Seven Oaks Dam, Pertinent Data

Item	Unit	
Drainage area	mi^2	177
Dam		
Crest elevation	ft, NGVD	2,610
Streambed elevation at dam centerline	ft, NGVD	2,060
Maximum height above streambed	ft	550
Crest length	ft	2,980
Freeboard	ft	5.6
Spillway		
Crest elevation	ft, NGVD	2,580
Crest length	ft	500
Elevation of maximum water surface	ft, NGVD	2,604.4
Outlet works (gated conduit) (1)		
Diameter of conduit (horseshoe section)	ft	18
Diameter of penstock	ft	11
Length of conduit	ft	1,623
High level intake elevation	ft, NGVD	2,265
Diversion intake elevation	ft, NGVD	2,100
Reservoir		
Area at spillway crest	acre	780
Gross capacity (at spillway crest)	acre-ft	145,600
Storage allocation below spillway crest		
Flood control	acre-ft	113,600
Sedimentation (100-year storage)	acre-ft	32,000
Reservoir Design Flood (general storm)		
Total volume (4-day)	acre-ft	115,000
Peak inflow	ft^3/sec	85,000
Peak outflow	ft^3/sec	7,000
Probable Maximum Flood (general storm)		
Total volume	acre-ft	356,000
Peak inflow	ft^3/sec	180,000
Peak outflow	ft^3/sec	174,000

NGVD denotes National Geodetic Vertical Datum of 1929.

(1) The outlet works downstream control alternative data is tabulated. See Appendix B for further details on other control alternatives.

CONTENTS

		Page
	Syllabus	iii
	Phase II GDM Listing of Volumes	v
	Pertinent Data	vii
	Contents	ix
I.	INTRODUCTION	I-1
	Authorization	I-1
	Scope and Purpose of Report	I-1
	Local Cooperation	I-1
II.	PROJECT PLAN	II-1
	General	II-1
	Description of Project Area	II-1
	Existing Flood Control Facilities	II-2
	The Flood Problem	II-2
	The Authorized Plan	II-3
	The Plan Recommended in this Report	II-7
	Consideration of Other Alternatives	II-8
	Dam Alternatives	II-8
	Outlet Works	II-8
	Spillway	II-9
	Energy Dissipator	II-9
	Relocations	II-10
	Powerhouse No. 2	II-11
	Flume Line	II-12
	Transmission Line	II-12
	Well No. 2	II-12
III.	PROJECT FEATURES	III-1
	General	III-1
	Dam Embankment	III-1
	Government Canyon Channel	III-2
	Outlet Works	III-2
	Energy Dissipator	III-2
	Spillway	III-3
	Access Roads	III-4

Contents (Continued)

		Page
	Reservoir Clearing	III-5
	Hydrologic Facilities	III-5
	Relocations	III-6
	Esthetic Treatment	III-7
	Administration Areas During and After Construction	III-7
IV.	HYDROLOGY	IV-1
	General	IV-1
	Design Criteria	IV-2
V.	HYDRAULIC DESIGN	V-1
	General	V-1
	Reservoir Area and Capacity	V-1
	Gross Capacity	V-1
	Sedimentation	V-1
	Volume	V-1
	Selection of Method of Distribution	V-2
	Tasks in Distributing Sediment	V-2
	Debris Pool	V-3
	Outlet Works	V-3
	Plunge Pool for Upstream Control Alternative	V-3
	Flip Bucket and Plunge Pool for Downstream Control Alternative	V-3
	General	V-3
	Final Energy Dissipating Alternatives	V-4
	Design Discharges	V-5
	Flip Bucket Design	V-5
	Radius of Curvature	V-5
	Minimum Deflection Angle	V-6
	Bucket Height	V-6
	Trajectories	V-7
	Throw Distance	V-7
	Impact Angle	V-7
	Scour Hole	V-7
	Jet Dispersion	V-7
	Scour Hole Depth	V-7
	Scour Hole Configuration	V-8
	Pre-Formed Plunge Pool	V-9
	Dimensions	V-9
	Excavation Volumes	V-10
	Cutoff Walls	V-10
	Depths	V-10
	Concrete Volumes	V-10
	Selection of Energy Dissipating Alternative	V-10
	Misoperation	V-10
	Selected Energy Dissipating Plan	V-11
	General	V-11
	Pre-Formed Plunge Pool	V-11
	Cutoff Wall	V-11

Contents (Continued)

	Page
Plunge Pool for Downstream Control Alternative	V-12
Model Study	V-12
Project Impacts	V-12
Flip Bucket and Plunge Pool for Mid-tunnel Control Alternative	V-12
Spillway	V-16
Need for Concrete Lining	V-16
Hydraulic Conditions	V-16
Other Considerations	V-16
Conclusions	V-17
Spillway Features	V-17
Crest Elevation	V-17
Alignment, Width, and Side Slopes	V-17
Approach Channel	V-17
Slope	V-17
Length	V-18
Spillway Design	V-18
Approach Channel Roughness	V-18
Spillway Rating Curve	V-18
Design Discharge	V-18
Downstream Channel Roughness	V-18
Downstream Channel Slope	V-19
Channel Velocities	V-19
Profiles	V-19
Freeboard	V-19
Freeboard Reference Level	V-19
Wave Runup	V-20
Wind Setup	V-22
Results	V-22
Minimum Revetment Design	V-23
General	V-23
Embankment Zones	V-23
Design Parameters	V-23
Design Wind	V-23
Effective Fetch	V-24
Design Wave	V-24
Significant Wave Height	V-25
Wave Runup	V-25
Layer Thickness and Stone Size	V-25
Design Procedure	V-25
Results	V-26
Diversion of Water During Construction for Upstream Control Alternative	V-27
Diversion of Water During Construction for Downstream Control Alternative	V-27
General	V-27
Diversion Tunnel	V-28
Elevation-Discharge Rating Curve	V-28
Inlet Control Analysis	V-28
Outlet Control Analysis	V-28

Contents (Continued)

	Page
Results	V-28
Elevation-Level of Protection Rating Curve	V-29
Reservoir Routings	V-29
Results	V-29
Steel Pipes	V-29
Elevation-Discharge Rating Curve	V-29
Energy Losses	V-29
Results	V-30
Elevation-Level of Protection Rating Curve	V-30
Reservoir Routings	V-30
Results	V-30
Steel Penstock	V-30
Elevation-Discharge Rating Curve	V-30
Inlet Control Analyis	V-30
Outlet Control Analysis	V-30
Results	V-31
Elevation-Level of Protection Rating Curve	V-31
Reservoir Routings	V-31
Results	V-31
Diversion of Water During Construction for Mid-Tunnel Control Alternative	V-31
Main Cofferdam	V-31
Government Canyon	V-32
Alternatives	V-32
Cofferdam and Pipeline	V-32
Cofferdam and Drainage Channel	V-32
Selected Design	V-33
Downstream Floodplain	V-34
Floodway Estimates	V-34
General	V-34
Prado Dam to Lytle-Warm Creek Confluence	V-34
Reach Description	V-34
Hydrology	V-34
Method of Analysis	V-35
Cross Sections	V-35
Bridges	V-35
Roughness Coefficients	V-36
Downstream Water Surface Elevations	V-36
Lytle-Warm Creek Confluence	V-36
Lytle-Warm Creek Confluence to Seven Oaks Damsite	V-36
Reach Description	V-36
Method of Analysis	V-37
Hydrology	V-37
Cross Sections	V-37
Bridges	V-37
Roughness Coefficients	V-38
Downstream Water Surface Elevations	V-38

Contents (Continued)

		Page
	Results	V-38
	Overflow Boundary Estimates	V-38
	General	V-38
	Method of Analysis	V-38
	Hydrology	V-38
	Results	V-39
	Sediment Transport	V-39
	References	V-39
VI.	GEOLOGY, SOILS, AND MATERIALS	VI-1
	General	VI-1
	Geologic Setting	VI-1
	Seismicity	VI-2
	Investigations	VI-3
	Foundation Conditions	VI-3
	Landslide Potential	VI-4
	Construction Materials	VI-4
	Design Considerations	VI-5
	Analysis	VI-6
	Construction Requirements	VI-7
VII.	STRUCTURAL DESIGN	VII-1
	General	VII-1
	References	VII-1
	Seismic Forces	VII-2
	Unit Design Stresses	VII-2
	Bridge Structure	VII-3
	Drop Structure	VII-3
VIII.	RELOCATION OF STREETS, RAILROADS, AND UTILITIES	VIII-1
	Powerhouse	VIII-1
	Flume Line	VIII-2
	Transmission Line	VIII-2
	Spreading Basins	VIII-3
	Well Number 2	VIII-3
IX.	ACCESS ROADS	IX-1
	General	IX-1
	Outlet Works Access	IX-1
	Dam Crest Access	IX-2
	Intake Structure Access	IX-3
	Spillway Access	IX-3
	Canyon Access	IX-3
	Geometric Design	IX-4
	Pavement Design Values	IX-4
	Drainage Structures	IX-4

Contents (Continued)

		Page
X.	SITE RESTORATION	X-1
XI.	RECREATION	XI-1
XII.	ENVIRONMENTAL ANALYSIS	XII-1
	Environmental Impacts	XII-1
	Sedimentation	XII-1
	Water Resources	XII-2
	Hydrology and Water Use	XII-2
	Water Quality	XII-2
	Air Quality	XII-2
	Land Use and Social Concerns	XII-2
	Important Farmlands	XII-2
	Recreation	XII-3
	Growth Inducement	XII-3
	Transportation and Utilities	XII-3
	Utilities	XII-3
	Access	XII-3
	Transport of Borrow Materials	XII-3
	Noise	XII-3
	Biological Resources	XII-3
	Cultural Resources	XII-4
	Mitigation	XII-4
	Biological Resources	XII-4
	Cultural Resources	XII-5
XIII.	DIVERSION AND CONTROL OF WATER DURING CONSTRUCTION	XIII-1
	Cofferdam	XIII-1
	Outlet Works	XIII-2
	Main Embankment	XIII-2
	Flood Emergency Plans	XIII-4
XIV.	REAL ESTATE REQUIREMENTS	XIV-1
XV.	FLOODWAY DELINEATION	XV-1
XVI.	COST ESTIMATES	XVI-1
	First Costs	XVI-1
	Operation and Maintenance	XVI-8
	Comparison of Estimates	XVI-8
XVII.	DESIGN AND CONSTRUCTION SCHEDULE	XVII-1
XVIII.	OPERATION AND MAINTENANCE	XVIII-1
	Operation and Maintenance Manual	XVIII-1
	Operation of Outlet Gates	XVIII-2
	Flood Emergency Plan	XVIII-2
	Water Quality Monitoring Program	XVIII-2
XIX.	RESERVOIR REGULATION	XIX-1

Contents (Continued)

Page

Tables

No.	Title	
V-1.	Hydraulic Parameters for Flip Bucket and Plunge Pool Analysis and Design for Downstream Control Alternative	V-5
V-2.	Dimensions of Ultimate Scour Hole With Flip Bucket for Downstream Control Alternative	V-8
V-3.	Dimensions of Ultimate Scour Hole Without Flip Bucket for Downstream Control Alternative	V-9
V-4.	Dimensions of Pre-formed Plunge Pool With Flip Bucket for Downstream Control Alternative	V-9
V-5.	Dimensions of Pre-formed Plunge Pool Without Flip Bucket for Downstream Control Alternative	V-9
V-6.	Hydraulic Parameters for Flip Bucket and Plunge Pool for Mid-Tunnel Control Alternative	V-13
V-7.	Dimensions of Ultimate Scour Hole With Flip Bucket for Mid-Tunnel Control Alternative	V-14
V-8.	Dimensions of Ultimate Scour Hole Without Flip Bucket for Mid-Tunnel Control Alternative	V-14
V-9.	Dimensions of Pre-formed Plunge Pool With Flip Bucket for Mid-Tunnel Control Alternative	V-15
V-10.	Dimensions of Pre-formed Plunge Pool Without Flip Bucket for Mid-Tunnel Control Alternative	V-15
VII-1.	Unit Design Stresses	VII-2
XVI-1.	Summary of First Costs	XVI-3
XVI-2.	Detailed Cost Estimate	XVI-4
XVI-3.	Comparison of First Costs	XVI-9
XVII-1.	Design and Construction Schedule	XVII-2
XVIII-1.	Annual Operation and Maintenance Costs	XVIII-3

Figures

No.	Title
1.	Reservoir Design Flood Inflow and Outflow Hydrographs Future Conditions
2.	Area-Capacity Curve
3.	Spillway Rating Curve
4.	Rating Curve Downstream and Midtunnel Control Outlet Tunnel
5.	Level of Protection provided by Downstream and Midtunnel Control Outlet Tunnel
6.	Rating Curve 3'-3" Steel Pipe
7.	Rating Curve 3'-6" Steel Pipe
8.	Level of Protection provided by 3'-3" and 3'-6" Steel Pipes Combined

Figures (Continued)

No.	Title
9.	Rating Curve 11' Steel Penstock
10.	Level of Protection provided by 11' Steel Penstock
11.	Rating Curve Upstream Control Outlet Tunnel with Access Gallery
12.	Level of Protection provided by Upstream Control Outlet Tunnel with Access Gallery

Plates

No.	Title
1.	Project Location
2.	General Vicinity and Borrow Area Map
2A.	Preliminary Land Requirements and Ownership Map
2B.	Biological Mitigation Lands
3.	Reservoir Plan
4.	General Plan
5.	Embankment Profile and Sections
6.	Embankment Sections and Details
7.	Spillway Plan, Profile and Section
8.	Spillway Hydraulic Profile
9.	Outlet Works, Downstream Control, Diversion/Outlet Tunnel Plan, Profile and Sections
10.	Outlet Works, Downstream Control, Downstream Structures
10A.	Upstream Control Plunge Pool Plan and Profile
10B.	Downstream Control Plunge Pool and Flip Bucket Plan and Profile
10C.	Midtunnel Control Plunge Pool and Flip Bucket Plan and Profile
11.	Outlet Works Access Road Plan and Profile Sta. 0+00 to Sta. 11+60
12.	Outlet Works Access Road Plan, Profile and Section Sta. 11+60 to Sta. 23+75
13.	Outlet Works Access Road Plan, Profile and Section Sta. 23+75 to Sta. 36+10
14.	Outlet Works Access Road Plan, Profile and Section Sta. 36+10 to Sta. 46+00
15.	Outlet Works Access Road Plan, Profile and Sections Sta. 46+00 to Sta. 54+06.87
16.	Alder Creek Road Improvement Plan and Sections
17.	Alder Creek Road Extension Plan, Profile and Sections
18.	Top of Dam to Spillway Access Road Plan, Profile and Details
19.	Spillway to Warm Springs Canyon Access Road Plan, Profile and Section Sta. 0+00 to Sta. 38+00
20.	Spillway to Warm Springs Canyon Access Road Plan, Profile and Section Sta. 38+00 to Sta. 72+60.16
21.	Top of Dam to Intake Structure Access Road Plan, Profile, Sections, and Detail

Plates (Continued)

No.	Title
22.	Reservoir Relocation Plan
23.	Cofferdam, Slurry Trench and Approach Channel Plan
24.	Existing Facilities and Haul Route Plan - Plate 1 of 3
25.	Existing Facilities and Haul Route Plan - Plate 2 of 3
26.	Existing Facilities and Haul Route Plan - Plate 3 of 3
27.	Reservoir Filling Frequency
28.	Reservoir Sediment Distribution
29.	Reservoir Sediment Profile
30.	Embankment Construction Schedule Below Streambed
31.	Embankment Rate of Placement above Streambed
32.	Drainage Basin
33.	Design Flood Peak Discharges Santa Ana River Future Conditions with Recommended Plan
34.	Preliminary 100-Year with Project Floodway and Floodplain Delineation - Plate 1 of 2
35.	Preliminary 100-Year with Project Floodway and Floodplain Delineation - Plate 2 of 2

Appendixes

No.	Title
A.	Geotechnical
B.	Seven Oaks Dam Outlet Works
C.	Emergency Action Plan During Construction
D.	Seven Oaks Dam Resource Use Master Plan
E.	Seven Oaks Dam Inundation Study
F.	Phase I Assessment, Embankment and Roller Compacted Concrete Dams
G.	Seven Oaks Dam Channel Stabilization Design and River Sediment Transport Study

I. INTRODUCTION

Authorization

1-01 Authorization of project construction is contained in the Water Resources Development Act of 1986, 99th Congress 2nd Session, P.L. 99-662. The project for flood control is contained in the report of the Chief of Engineers for the Santa Ana River Mainstem, including Santiago Creek, California, dated January 15, 1982, except that, in lieu of the Mentone Dam feature of the project, the Secretary is authorized to plan, design, and construct a flood control storage dam on the upper Santa Ana River. The full authorization language is presented in the Main Report.

Scope and Purpose of Report

1-02 This volume of the Phase II General Design Memorandum (GDM) describes the latest existing physical and environmental conditions in the project area and provides definite design for Seven Oaks Dam. The design presented here will be refined in future Feature Design Memoranda (FDM). This Phase II GDM provides the basis for determining the project rights-of-way and easements, updating the project costs, assessing the environmental effects, and preparing subsequent FDMs.

Local Cooperation

1-03 The division of Federal and non-Federal responsibilities for local cooperation are outlined in the Main Report.

II. PROJECT PLAN

General

2-01 The Santa Ana River Basin drains approximately 2,450 square miles (mi^2), excluding an area of 32 mi^2 tributary to Baldwin Lake and 10 mi^2 tributary to Perris Reservoir. Of the total basin, 2,255 mi^2 of the drainage area is upstream from Prado Dam, which is the major flood control structure on the Santa Ana River. Approximately 23 percent of the basin is within the San Gabriel and San Bernardino Mountains; about 9 percent is in the San Jacinto Mountains; and 5 percent is within the Santa Ana Mountains. Most of the remaining area is in the valleys formed by the broad alluvial fan along the base of these mountains (pl. 1). Maximum elevation at San Antonio Peak in the San Gabriel Mountains is 10,080 feet above the National Geodetic Vertical Datum of 1929 (NGVD); at San Gorgonio Peak in the San Bernardino Mountains, 11,502 feet (NGVD); and at Mount San Jacinto in the San Jacinto Mountains, 10,804 feet (NGVD). The San Bernardino Mountains are the source of the Santa Ana River and of two of its principal tributaries, Bear and Mill Creeks. The Santa Ana River has an average gradient of about 240 feet per mile (ft/mi) in the mountains.

Description of the Project Area

2-02 The upper Santa Ana River drainage area for Seven Oaks Dam is 177 mi^2, excluding the 32 mi^2 tributary to Baldwin Lake, and has its headwaters in the San Bernardino Mountains. Elevations vary from 10,664 feet NGVD at Anderson Peak and 11,502 feet NGVD at San Gorgonio Peak to 2,060 feet NGVD at the damsite, which is approximately 1 mile upstream from the canyon mouth. Generally trending southwesterly, the 27 miles of river upstream from the site have an average gradient of 300 ft/mi. However, small tributaries originating in the high mountains have gradients that exceed 1,900 ft/mi. Bear Creek, the principal tributary within the canyon area, drains 55 mi^2 and has an average gradient of approximately 460 ft/mi.

2-03 Urban development exists downstream from the canyon mouth, with the cities of Mentone and Redlands, less than 8 miles away. Development within the canyon is limited to a hydropower system constructed at the turn of the century and owned by the Southern California Edison Company, and to irrigation and water facilities primarily operated by the San Bernardino Valley Municipal Water District. There are a few scattered residential developments in the upper canyon area.

Existing Flood Control Facilities

2-04 The only existing facility which could affect floodflows in the Upper Santa Ana River Canyon is Big Bear Lake Dam, a recreation and water supply reservoir, located upstream from the recommended damsite. The lake, owned by Big Bear Municipal Water District, controls 38 mi^2 and has a surcharge storage of about 8,600 acre-feet at the top of dam. There are numerous diversion structures and other water supply and hydropower related facilities in the canyon. Downstream from the canyon mouth on the alluvial fan, there are various dikes and water spreading basins constructed for water conservation and groundwater recharge purposes by the San Bernardino Water Conservation District.

The Flood Problem

2-05 The Santa Ana River is currently uncontrolled for much of its length in San Bernardino and Riverside Counties down to Prado Dam. Prado Dam is the only flood control reservoir along the 65-mile length of the Santa Ana River. The area upstream from Prado Dam is threatened by major storms. In Riverside County, the 1969 floods seriously damaged two sewage treatment plants, ruptured numerous sewage lines, washed out several bridges and bridge approaches, flooded large areas of agricultural land, and caused heavy bank erosion along most of the river. In San Bernardino County, there was widespread overflow of much of the 100-year floodplain, erosion of levees, and damage to county roads and utilities. One county bridge was washed off its supports. Damages were also significant on several major tributaries to the Santa Ana River. Floods on Temescal Wash and San Timoteo Creek covered residential, commercial, and agricultural property with several feet of mud and sand.

2-06 Construction of Prado Dam was completed in May 1941. Based on current hydrologic criteria, Prado Dam provides only about 70-year flood protection to downstream communities. The Standard Project Flood (SPF) would inundate over 160 mi^2 of highly urbanized land in Orange County. Parts of the downstream channel near Fountain Valley and Huntington Beach can contain less than a 50-year flood.

2-07 The seriousness of the problem was illustrated during the storms of 1980. Following 10 days of almost continuous rainfall resulting from a series of six storms, the water level in Prado Reservoir reached elevation 528 feet NGVD, the highest on record. Flood releases from Prado Dam were increased to 20 percent of the downstream channel rated

capacity. Although the channel should have contained these flows safely, the channel invert eroded badly, and many of the channel side slopes failed along a 4-mile reach in the City of Santa Ana. Emergency flood-fighting efforts prevented major levee breaks, but outflows from Prado Dam had to be reduced to accomplish emergency efforts. This reduction left the reservoir 60 percent full and vulnerable to additional storms that fortunately did not materialize. This experience demonstrated the weaknesses in the existing system:

a. Prado Dam could not provide acceptable levels of protection to highly urbanized downstream areas.

b. Downstream earth-bottom channels were not capable of carrying the flows over a sustained period of time.

2-08 The flood problem on the Santa Ana River Mainstem is a basin-wide problem. It is necessary to protect heavily developed communities downstream from Prado Dam and to provide protection for the rapidly growing upstream communities. Studies have also concluded that the only acceptable flood control improvement plan to protect against the largest flooding considered reasonable to the area would involve:

a. Enlarging the existing channel downstream from Prado Dam.

b. Enlarging Prado Reservoir, but only to the extent that resulting socioeconomic effects are held to acceptable levels.

c. Providing additional flood storage upstream from Prado Dam.

The Authorized Plan

2-09 The Santa Ana River Phase I GDM submitted to Congress in September 1980 recommended in addition to other flood control features, the construction of a flood control dam, (Mentone Dam), on the Santa Ana River. Mentone Dam was to be located just northwest of the City of Mentone and north of the City of Redlands, California. The estimated cost for Mentone Dam was $530,032,000 (October 1985 price levels). Due to considerable opposition to the dam based on project effects to existing water recharge facilities in the region, and esthetic considerations, the Assistant Secretary of the Army (Civil Works) recommended that the Santa Ana River Mainstem Project be authorized "with the exception of Mentone Dam."

2-10 Congress directed the Corps to study alternatives to the previously proposed Mentone Dam under Section 1304 of the 1984 Supplemental Appropriations Bill. In accordance with further directives, the study was focused on upstream flood storage alternatives. Local interests also favored a study of upstream flood storage alternatives.

2-11 The Upper Santa Ana River Flood Storage Alternative Study, Supplement to Phase I GDM on the Santa Ana River Mainstem including Santiago Creek was completed in December 1985. The Upper Santa Ana River Dam, now called the Seven Oaks Dam, was the recommended alternative to Mentone Dam and was subsequently authorized. The recommended plan consisted of a dam in the Santa Ana River Canyon about 4 miles upstream from the previously proposed Mentone damsite and about 8 miles northeast of the City of Redlands. The dam would have been an earth and rock fill structure with a height of about 550 feet above the existing streambed, crest width of 40 feet, crest length of about 3,000 feet, and crest elevation at 2,610 feet NGVD. The upstream and downstream slopes would have been 1 vertical on 2 horizontal (1V on 2H). The dam would have had a storage capacity of 160,000 acre-feet at the spillway crest elevation of 2,580 feet NGVD. The storage was allocated as follows:

a. Sediment: 32,000 acre-feet

b. Flood Control: 128,000 acre feet

The dam would have provided approximately 88 percent of the storage capacity of the previously proposed Mentone Dam.

2-12 A detached spillway was to be located about 1,700 feet east of the dam. It would have had a trapezoidal cross section with a base width of 500 feet and side slopes averaging 1V on 1H. The spillway would have been unlined except for a concrete control sill across the invert at the crest.

2-13 The outlet works tunnel was to be located within the east abutment. The tunnel, excavated in rock and lined with reinforced concrete, would have been approximately 1,900 feet long with a finished inside diameter of 15 feet. A vertical control shaft would have been constructed to intersect the outlet tunnel approximately one-third the distance downstream from the intake where a gate chamber would be constructed. The outflow would have been regulated by three sets of 8-foot-wide by 10-foot-high tandem vertical lift gates; each set with an operation gate and an emergency gate. The gates were to be housed in a transition section at the base of the control shaft. The control shaft would have been a reinforced concrete structure with a height of 530 feet and a circular cross section 24 feet in diameter. The permanent intake sill elevation was to be at 2,180 feet NGVD. The intake structure was to be sloped up at the abutment and was to incorporate a sliding bulkhead gate and moveable trash rack system. The invert elevation at the downstream end of the outlet would have been at 2,050 feet NGVD. The outlet was to end in a flip bucket into a grouted stone plunge pool approximately 730 feet long and 320 feet wide.

2-14 The proposed foundation treatment required a triple-line grout curtain, with injection holes spaced approximately 10 feet apart. The center line of the grout curtain was to be about 300 feet deep under the maximum section of the dam and decreasing with elevation in the abutment area.

2-15 A drainage channel was to be excavated through the ridge separating Government Canyon from the Santa Ana River. The drainage channel would enable flows from Government Canyon to drain into the Santa Ana River and prevent a separate impoundment at Government Canyon.

2-16 The existing hydropower facility in the reservoir-dam area is the Southern California Edison's (SCE) system of flumes and three hydropower plants. Within the project area are Powerhouse No. 2, (the second of three powerhouses in the series), an access road, flumes, and transmission lines. It was proposed to waterproof sensitive equipment at Powerhouse No. 2 and provide payment for periodic inundation so that it could remain in place. A 1-mile-long section of open flume was to be structurally modified with concrete supports to withstand periodic inundation and electrical transmission lines leading from the powerhouses relocated above the maximum reservoir pool.

2-17 Forest Service access roads and a water well owned by the Bear Valley Mutual Water Company would also have been affected by the project. The well was to be left in place, but replaced by a submersible pump with a waterproof power system. An alignment requiring about 2 miles of new road was to extend to the top of the dam from the existing Alder Creek Road, near the dam's west abutment, and then upstream onto the canyon floor, where it would have reconnected with the existing access roadways to upstream Forest Service lands and to Southern California Edison facilities. About 1,000 acres of Forest Service lands would have been required for flood control purposes.

2-18 The downstream Santa Ana River streambed was the proposed pervious borrow source for dam construction. A preferred area for pervious borrow materials was located in an area that roughly corresponds to the existing water conservation spreading grounds just south of Greenspot Road (pl. 2). Use of this site would have involved removing surface alluvial materials from the area; as well as recontouring the site following completion of borrow operations and reconstruction of spreading facilities.

2-19 A recreation plan for the Seven Oaks Dam was prepared which consisted of an access road, a trail for hiking and equestrian use, and a day-use site providing trail access, as well as picnic, barbecue, restroom, and parking facilities. The quality of the recreation experience in the canyon was to be changed from that associated with natural, wilderness-type environments to a more urban/rural-oriented development.

2-20 The introduction of Corps project-related recreational development was to supplement the recreation investments planned by the U.S. Forest Service within the watershed. The Forest Service requested consideration of "in lieu" site recreation which would allow them to shift the planned investment or comparable future resources for recreational development from the project site to other more accessible Forest Service sites. However, further study of "in lieu" site recreation was not undertaken due to its inconsistency with the Corps of Engineers policy on recreational development.

2-21 The recommended plan called for extensive mitigation of environmental impacts. The mitigation plan for long-term impacts to wildlife habitat was incrementally evaluated to ensure that the recommended features met mitigation goals in the most cost-efficient manner possible. Gains in environmental value associated with individual mitigation measures were evaluated in cooperation with U.S. Fish and Wildlife Service, U.S. Forest Service, and the California Department of Fish and Game. The mitigation evaluation was described in the SEIS accompanying the Phase I Supplemental GDM.

2-22 The mitigation element of the recommended plan for biological resources had six main features:

a. Acquisition of Section 5, an area of upland habitat immediately adjacent to the damsite. Purchase of this 649-acre site would have prevented it from being developed (a direct effect of improved access to the area as a result of the project), and would thus have preserved a significant deer habitat area and minimized project-associated impacts.

b. Acquisition of Filaree Flats, currently owned by the Bear Valley Mutual Water Company. Acquisition of this 138-acre site would have provided for very cost effective preservation of high quality riparian, aquatic, and upland habitat.

c. Habitat improvement of USFS lands on Bear Creek were to include revegetation, recontouring, slope stabilization, and improved USFS management. These improvements were to provide significant riparian, aquatic, and upland habitat for deer.

d. Improvement of public lands in the Santa Ana Wash, which would provide a considerable area of riparian and aquatic habitat.

e. Protection of the Federally endangered Eriastrum (plant species) within the Santa Ana Wash that would be threatened by the project to assure no net adverse effects.

f. Improvement (reseeding) of areas temporarily affected by construction activities.

2-23 It should be noted that there were trade-offs between upland and more valuable riparian habitat involved in this plan. The mitigation plan was developed in coordination with resource agencies. It was recommended by the Chief of Engineers contingent upon confirmation of results during detailed design phases.

2-24 Impacts to cultural sites would also have been mitigated. The grave sites of an infant and stillborn child in the vicinity of the outlet works required relocation.

2-25 For the 35-mile reach extending from Seven Oaks Dam to Prado Dam a 100-year floodway was to be delineated. The floodway was to be evaluated in accordance with the Federal Emergency Management Agency (FEMA)

guidelines, except that flows under future conditions would be utilized with Seven Oaks Dam in place and operational. The local sponsors were to have the responsibility to manage the floodway and fringe floodplain in accordance with regulations established by FEMA.

The Plan Recommended in this Report

2-26 The plan recommended in this report differs very little from the authorized plan. Seven Oaks Dam remains an earth and rock fill structure with approximately the same dimensions as the authorized plan. The storage volume afforded by the dam is smaller due to changes in the characteristics of the reservoir based on more detailed topographic mapping. The reservoir's storage volume has been reduced from 160,000 acre-feet to about 145,600 acre-feet at the spillway crest elevation of 2,580 feet NGVD (pl. 27).

2-27 The sediment storage allocation in the reservoir is 32,000 acre-feet; however, detailed sediment distribution studies using the current topographic mapping revealed that Southern California Edison Company's Powerhouse No. 2 would be buried by approximately 50 feet of sediment at the end of project life (pls. 28 and 29). This finding impacted the relocation alternatives previously considered, which only addressed potential inundation impacts. A relocation plan was identified for the Edison facilities based on consideration of the sediment conditions. For estimating costs of the relocation, a plan consisting of removing Powerhouses Nos. 2 and 3, replacing the flume between Powerhouse No. 2 and 3 with a pressure pipe, replacing powerhouse No. 3 to accommodate the higher head, and relocating the transmission line above the proposed maximum reservoir level was evaluated (pl. 3).

2-28 The estimated depth of sediment accumulation at the intake of the outlet works increased from approximately 80 feet to 165 feet. The increase in the depth of sediment required the modification of the outlet works. In this volume of the Phase II GDM Report, three outlet works schemes are presented. Also see Appendix B for a detailed description of all three schemes. The first scheme provides for an upstream tunnel control and consists of a high level intake tower with control gates, a regulating diversion/outlet tunnel with an overhead access gallery, a downstream access and equipment structure, and an outlet channel connecting the tunnel downstream portal and an energy dissipating plunge pool. The second system provides for a downstream tunnel control and consists of a high level intake tower, a regulating diversion/outlet tunnel, a pressurized steel regulating outlet conduit inside the tunnel, a downstream control and equipment structure located at the tunnel outlet portal, and an outlet channel connecting the control structure to an energy dissipating plunge pool (pls. 9 and 10). The third scheme, consists of a mid-tunnel control chamber, with access to the vaulted gate chamber from a vertical access shaft. The downstream control tunnel alternative is the recommended plan. For the purpose of estimating the cost of the project, the downstream control alternative is presented in the project cost estimates (Section XVI).

2-29 The mitigation plans have changed slightly from the authorized plan. The acquisition of Filaree Flats remains as part of the mitigation plan; however, due to the over compensation of habitat losses, the least cost efficient feature of the mitigation plan, Bear Creek mitigation area, was deleted. Opportunities for recreational development were evaluated, but recreation facilities are no longer recommended because they would be inconsistent with current management objectives of the United States Forest Service.

2-30 The Southern California Edison Santa Ana River Hydroelectric System has been determined eligible for listing in the National Register of Historic Places (NRHP), and significant elements in the district such as the powerhouses, flumes and transmission lines, if impacted, will be mitigated. Other properties, if determined NRHP elibible will require mitigation. Mitigation measures will be formulated in consultation with the Advisory Council on Historic Preservation and the State Historic Preservation Officer. Mitigation measures may involve documentation of historic structures, the excavation of significant archeological sites for data recovery, or avoidance/preservation measures where feasible.

Consideration of Other Alternatives

2-31 Alternatives were evaluated for the dam type, the outlet works, the spillway, the energy dissipator, and relocation of Southern California Edison Company facilities. A summary of the alternatives considered for each feature and the basis for selecting the recommended plan is presented in the following paragraphs.

DAM ALTERNATIVES

2-32 The two alternatives evaluated for the dam were: (1) a Roller Compacted Concrete (RCC) dam, and (2) an earth and rock fill dam. Preliminary cost studies indicated that both methods of construction would be competitive at this site with potential cost savings being attributed to the RCC dam. However, due to the proximity of the dam to the San Andreas fault and other smaller faults and trace faults, a recommendation was made by Dr. Clarence Allen, Engineering Seismologist at the California Institute of Technology, that the dam be designed to accommodate 4 feet of displacement at any location in the foundation rock and in any direction. Given this criteria, unprecedented defensive measures were incorporated for the RCC alternative and subject to review by geotechnical and structural experts. Based on the expert review, the earth and rock fill dam was selected as the preferred alternative. A technical assessment for the earth and rock fill dam, the RCC dam, and expert review and comment are included as Appendix F to this volume.

OUTLET WORKS

2-33 Three outlet works alternatives were evaluated for the Phase II GDM study: (1) upstream control with access from the downstream end of the tunnel through an overhead gallery, (2) downstream control with

service gates at the downstream end of the tunnel, and (3) mid-tunnel control with access to a vaulted gate chamber from a vertical access shaft (Appendix B). Each of the options appear to be competitive from a cost standpoint. The recommended outlet works is the downstream control alternative.

SPILLWAY

2-34 Three alternative spillway designs were evaluated during the Phase II GDM study: (1) the design developed during the Phase I GDM, modified to include concrete lining on the downstream slope; (2) an unlined spillway with a horizontal approach and an ogee control section at the downstream end, and (3) a horizontal unlined spillway.

2-35 Geotechnical investigations at the spillway during the Phase II GDM study indicated that the rock mass is highly fractured and moderately weathered. Consequently, some erosion of the spillway channel will occur during spillway flows. The possibility of gully formation back to and undercutting the recommended concrete control sill is considered to be unlikely, even during PMF flows. Therefore, because of (1) the infrequency of use, (2) the short duration of spillway flows, (3) the deepening and widening of the concrete control sill at an upstream location to ensure spillway control, and (4) the disproportionate costs of a concrete-lined spillway compared to the possible damages averted, the Phase I design with slight modification would be implemented.

2-36 The original Phase I spillway design had an adverse slope to the crest of 0.04 and a slope downstream of the crest of 0.02. A 6-foot deep concrete control sill was located across the invert at the spillway crest 700 feet from the downstream end of the spillway. The spillway design was refined in the Phase II GDM to increase the distance from the crest to the downstream end so that, if a headcut were to develop, a longer period of time would be required for the headcut to reach the concrete control sill. The concrete control sill is now located approximately 1,000 feet from the downstream end of the spillway (pl. 8). In addition, the width and depth of the concrete sill was increased from 8 to 11 feet and 6 to 10 feet respectively.

ENERGY DISSIPATOR

2-37 Ten alternatives and features for the energy dissipator downstream from the tunnel outlet were developed on a conceptual basis, however, only three alternatives were considered to be viable. The evaluation and details of the alternatives are provided in Appendix G. The ten alternatives and features and the conclusions reached on each are presented below.

a. Do nothing. Due to the possibility of a scour hole undermining the outlet portal and redistribution of streambed materials during flows, this concept was dropped from further consideration.

b. Protection of the downstream end of the outlet portal. This alternative, consisting of constructing a concrete cutoff wall, was carried forward in the study.

c. Flip bucket. This alternative was carried forward in the study.

d. Flip bucket discharging into a pre-formed plunge pool created by excavating the alluvium. This alternative was carried forward in the study.

e. Energy dissipator with submerged flow. This alternative was dropped from further consideration due to the large depth of excavation required.

f. Submerged bucket. This alternative was dropped from further consideration due to the large depth of excavation required.

g. Channel stabilizers. This alternative was dropped from further consideration due to the natural self-armoring ability of the streambed.

h. Drop structures. This alternative was dropped from further consideration due to the natural self-armoring ability of the streambed.

i. Channelization. This alternative was dropped as unnecessary.

j. Site specific erosion and flood protection. This alternative was dropped from further consideration because the sediment transport studies indicated that the channel would armor before any significant degradation would occur.

2-38 The selected energy dissipator for the downstream and mid-tunnel outlet works alternatives consists of constructing a cutoff wall downstream from the outlet channel and a flip bucket discharging into a pre-formed plunge pool. The selection was based on cost considerations. The flip bucket alternative was estimated to be less expensive than the without flip bucket plan due to the shorter cutoff wall required. For the upstream control alternative, the without flip bucket alternative was less costly. Elimination of the pre-formed plunge pool was also considered, but the need to provide adequate initial energy dissipation and to prevent large sediment deposits in the downstream channel ultimately led to the decision to retain the pre-formed plunge pool.

RELOCATIONS

2-39 Relocation alternatives were developed for four primary features:

a. Southern California Edison (SCE) Powerhouse No. 2

b. SCE flume line between Powerhouses Nos. 2 and 3

c. SCE transmission line

d. Bear Valley Mutual Water Company Well No. 2

Seven alternatives were developed for SCE Powerhouse No. 2. Of the seven alternatives, four required continued powerhouse operation and the necessity to relocate the existing flume line. Two alternatives were developed for the flume line relocation. In addition six of the seven powerhouse relocation alternatives required relocation of the existing transmission lines. Two alternatives were evaluated for the transmission line relocation. Relocation of Well No. 2 is required independent of the other relocations. Two alternatives were evaluated for the well relocation. The alternatives considered for each of these features is presented in the following paragraphs.

POWERHOUSE NO. 2

2-40 The relocations or modifications were reanalyzed to account for the changes in the area capacity data and the sediment distribution. The purpose of the study was to develop a reasonable cost estimate for feasibility evaluation. There are a number of relocation/modification options that appear competitive. The relocation/modification study results are presented in a report entitled "Seven Oaks Dam, Final Concepts Report, Preliminary Cost Estimate for Relocations/Modification to SCE Facilities" (November 1987, by Robert Bein, William Frost, and Associates). The following seven relocation/modification alternatives were considered for the system:

a. Construct a watertight, protective concrete bunker over the existing Powerhouse No. 2 facilities.

b. Eliminate Powerhouse No. 2 and increase the generating capacity at Powerhouse No. 3 downstream to accommodate the additional head.

c. Eliminate Powerhouse No. 2, and pay SCE damages for lost power from the wasted head.

d. Floodproof Powerhouse No. 2 for less than SPF protection and pay SCE damages after the facility becomes inoperable due to inundation and sediment impacts.

e. Eliminate Powerhouses No. 2 and No. 3 and pay SCE damages for lost power from the wasted heads.

f. Provide pumped storage above Powerhouse No. 1 for use during peak demand and eliminate Powerhouses No. 2 and No. 3 and pay SCE damages for the net lost power in the system.

g. Purchase the entire system and lease back the operations to the Edison Company for as long as possible.

Two cost competitive solutions were the option to provide a protective bunker at Powerhouse No. 2 (a above) and the option to eliminate Powerhouse No. 2, transfer the head to Powerhouse No. 3 and install new turbines (b above). The option to eliminate Powerhouse No. 2 and to modify Powerhouse No. 3 to accommodate the additional head was selected as the preferred solution because it could provide equivalent energy production. The cost for the Powerhouse No. 3 modification option was estimated at $12,150,000. The option to provide a protective bunker was approximately the same cost, but contained operational difficulties (including periodic inundation) which were undesireable. The other options were estimated to cost over $15,000,000.

FLUME LINE

2-41 The two relocation alternatives for the flume line between Powerhouses No. 2 and No. 3 were: (1) replace the open channel portions of the flume with a closed conduit, concrete-line the existing tunnel portions and maintain the existing alignment, and (2) replace the flume with a closed, pressurized conduit and realign it along the streambed. Replacement of the flume with a closed pressurized conduit located in the streambed was the selected option based on advantages in constructability and compatibility with the selected powerhouse relocation alternative. The conduit will be constructed by tunneling through the left abutment at the damsite near the dam's outlet works. The conduit through the left abutment will be a 48-inch diameter steel pipe separated from the outlet works. It will be located approximately 200 to 300 feet east of the dam's outlet works tunnel and will have a parallel alignment.

TRANSMISSION LINE

2-42 The relocation alternatives for the transmission line were limited to evaluating an alignment above the reservoir pool on the east rim of the reservoir as opposed to the alignment shown on the west rim of the reservoir in the Phase I Supplemental GDM. The alignment on the east rim of the reservoir was favored by the United States Forest Service on the basis of esthetics and by the Southern California Edison Company on the basis of constructability. The east rim alignment was selected on the basis of constructability on more favorable terrain.

WELL NO. 2

2-43 Two relocation alternatives were examined for the Bear Valley Mutual Water Company Well No. 2. One alternative consisted of replacing the existing electric pump with a submersible unit and relocating the pump control panels above the projected reservoir pool. The other alternative consisted of abandoning the existing well and drilling a new well downstream from the proposed damsite and installing a new pump and motor. The option to replace the existing well with a submersible unit was viewed to have more potential for operation and maintenance problems. These problems would be created by sedimentation at the existing well in addition to difficult access to the high level controls. Therefore, the selected plan calls for a new well in the vicinity of Powerhouse No. 3.

III. PROJECT FEATURES

General

3-01 The following paragraphs describe each of the flood control features of the project. The relocation of utilities, existing access roads, and the permanent access roads to the project are covered in more detail in sections VIII and IX of this volume.

Dam Embankment

3-02 Seven Oaks Dam will be constructed with earth and rockfill materials from spillway and drainage channel excavation and nearby alluvial deposits. The dam will be approximately 630 feet above the lowest foundation bedrock contact and approximately 550 feet above the existing streambed at the dam axis. The crest elevation of the dam will be at 2,610 feet NGVD with a crest length of 2,980 feet. The crest width will be 40 feet. The outer slopes of the embankment will be 1V on 2H (pl. 4)

3-03 The embankment section will be designed to produce a conservatively safe dam while utilizing the materials available at the site in the most cost-effective manner. The impervious central core will extend from bedrock to within 10 feet of the crest of the dam (pl. 6). The top width of the core will be 16 feet and the base width will be approximately one-third of the maximum hydrostatic head. The core will be flanked by upstream and downstream filter zones composed of minus 3-inch alluvial materials processed from the downstream pervious borrow area or from the foundation excavation. The upstream shell will consist of free-draining minus 12-inch alluvial materials processed from the downstream pervious borrow area, with an outer layer of oversize stone. The downstream transition zone will consist of unprocessed rock excavated from the spillway and Government Canyon Ridge, with an outer layer of oversize stone from the pervious borrow area.

3-04 Embankment zoning provides resistance to concentrated leaks by placing high strength and erosion resistant materials at the outer shells. A crack-resistant impervious core and well-graded, cohesionless filter and transition zones are designed to be effective in sealing internal cracks which may develop due to differential settlement or earthquake induced movement. The outer shells of free-draining sands, gravels and rockfill are designed with relatively flat slopes to provide stability during an earthquake.

Government Canyon Channel

3-05 A channel to connect Government Canyon with the main Santa Ana River will be excavated through Government Canyon Ridge to permit drainage of the Government Canyon watershed during diversion and control of water (pl. 23). The initial drainage channel excavation will create a trapezoidal notch through Government Canyon Ridge with a minimum bottom width of 30 feet, side slopes of 1V on 1H, and an invert slope from Government Canyon to the Santa Ana River of about 0.05 over a 700-foot length. The drainage channel in conjunction with a 50-foot high cofferdam will provide greater than 25-year protection.

3-06 The drainage channel excavation on Government Canyon will later be expanded as the embankment construction proceeds. The excavated materials will be used in the downstream transition zone of the embankment (pls. 5 and 6). Approximately 4,300,000 yds^3 of material will be excavated from the ridge, (800,000 yds^3 as part of the initial drainage channel diversion and 3,500,000 yds^3 of expanded excavation as required for embankment construction). After the required volume of material is excavated, the area will be graded for drainage into the Santa Ana River, and the initial drainage channel will no longer exist.

Outlet Works

3-07 Three complete designs (upstream control with downstream access through an overhead gallery, downstream control, and mid-tunnel control with a vertical access shaft) are presented in Appendix B of this volume, entitled "Seven Oaks Dam Outlet Works." The recommended plan is the downstream control alternative. The recommended outlet tunnel will be excavated through the east (left) abutment. The tunnel will be used during the embankment construction for diversion of streamflows around the embankment site. With a 50-foot-high cofferdam, the diversion tunnel will provide 18-year flood protection to below streambed foundation excavation and embankment construction activities. A detailed description of the diversion and control of water is covered in section XIII.

Energy Dissipator

3-08 The energy dissipator associated with the downstream tunnel control and mid-tunnel control alternatives consist of a flip bucket with pre-formed plunge pool and toe protection for the outlet channel. Normal operation releases will discharge from the flip bucket. Under

these conditions, a scour hole would be formed by the resulting water jet, eventually providing the required energy dissipation. The alignment and angle of the flip bucket will determine the location of the scour hole. The upstream control alternative was found to be less costly without the flip bucket.

3-09 The pre-formed plunge pool will be armored with cobbles and boulders removed from the native alluvium. The initial plunge pool for the downstream control alternative will have a top length of approximately 790 feet. The top width will vary from approximately 130 to 460 feet. The maximum depth will be approximately 14 feet. The side slopes will be excavated to 1V on 3H. The armoring will consist of stones larger than 6 inches in size. The pre-formed plunge pool is expected to scour down deeper under project operation until stability is attained at approximately 21 feet deep. The initial plunge pool for the upstream control alternative will have a top length of approximately 330 feet. The top width will vary from approximately 190 to 300 feet. The maximum depth will be approximately 23 feet. Project operation stability will be attained at approximately 34 feet deep. For the mid-tunnel control alternative, the initial plunge pool will have a top length of approximately 540 feet. The top width will vary from approximately 160 to 360 feet. The maximum depth will be approximately 16 feet. Project operation stability will be attained at approximately 24 feet deep. The final top dimensions of the plunge pools are expected to remain the same as the pre-excavated dimensions (pls. 10A, 10B, and 10C). The pool alignment and size are subject to change pending detailed studies to better estimate actual scour depths and scour hole configurations at the feature design level. Also, different flip bucket angles will be studied to better align the pool away from the outlet works access road.

3-10 A cutoff wall will be provided at the downstream end of the outlet works exit channel to provide scour protection to the flip bucket, exit channel and outlet portal. Details of the exit channel and cutoff wall are presented in Appendix B. The cutoff wall for the downstream control alternative is inclined on a slope of 1V on 1H and flared with a top width (just under the flip bucket) of 24 feet and a bottom width of 169 feet. The toe of the cutoff is at elevation 2018 feet NGVD, approximately 30 feet below the exit channel invert. The cutoff wall for the upstream control alternative has a top width of 18 feet and a bottom width of 215 feet. The toe of the cutoff is at elevation 2008 feet NGVD, approximately 40 feet below the exit channel invert. For the mid-tunnel control alternative, the cutoff wall has a top width of 18 feet and a bottom width of 120 feet. The toe of the cutoff is at elevation 2009 feet NGVD, approximately 40 feet below the exit channel invert.

Spillway

3-11 The spillway, 500 feet wide and approximately 1400 feet long, will be excavated in rock in a natural saddle just east of the dam (pl. 7). The spillway section will be trapezoidal and unlined except for a concrete control sill located approximately 1,000 feet from the down-

stream end. The control sill will be of cast-in-place, reinforced concrete construction. The top of the control sill will be at elevation 2,580 feet NGVD. The invert slope leading up to the sill will be on an adverse grade of 0.025. The invert slope downstream from the sill will be 0.02. The side slopes will be an average of 1V on 1H, though the actual side slopes will have benches 25 feet wide, at 50-foot vertical spacings with side slopes between benches excavated to 1V on 0.5H.

3-12 The top of the spillway excavation daylights 370 feet above the crest at approximately elevation 2,950 feet NGVD. The peak water surface elevation during a Probable Maximum Flood (PMF) event is estimated at elevation 2,604.4 feet NGVD. The water surface profile through the spillway is shown on plate 8.

Access Roads

3-13 Paved all-weather access roads will be provided to the outlet works, dam crest, intake structure, and spillway. Permanent access within the reservoir and upstream canyon will be provided by a graded, unpaved road equivalent to the existing unpaved access road in the streambed. Also see section IX, Access Roads.

3-14 The access road to the downstream portal of the outlet works will extend from Greenspot Road, a distance of approximately 5,500 feet. The road will generally parallel the west bank of the Santa Ana River (pls. 11 through 15) before crossing over to the east bank near the existing SCE Powerhouse No. 3 and continuing up the canyon to access the outlet works.

3-15 The access road to the crest of the dam will consist of, first, improving approximately 18,350 feet of the existing unpaved Alder Creek Road from Greenspot Road by widening and paving (pl. 16) and, second, constructing a new extension from the existing Alder Creek Road to the top of the dam at the west abutment approximately 3,700 feet in length (pl. 17). The intake structure access road (pl. 21) will extend from this point and along the west abutment to the upstream face of the dam. The road will then be constructed along the upstream embankment face at about a 6 percent grade. The road will then traverse part of the east abutment until it terminates at the intake structure, a total distance from the top of the dam of approximately 4,785 feet. Access to the intake structure vertical tower will be provided by a reinforced concrete bridge. The spillway access road (pl. 18) will extend from the road on the top of the dam at the east abutment and follow the reservoir rim to the spillway location, a distance of approximately 1,775 feet.

3-16 The upstream canyon access road will branch off from the spillway access road and traverse the upstream edge of the spillway. From here the unpaved road will follow the reservoir rim until it meets the existing unpaved access road in Warm Springs Canyon, a distance of approximately 7,260 feet (pls. 19 and 20). This existing road can then be used to enter the upper Santa Ana River canyon.

Reservoir Clearing

3-17 Generally, the extent of reservoir clearing is evaluated based on the flood control operational needs of the dam. Clearing the reservoir will consist of removing all floatable material in the debris pool area below the elevation, 2,299 feet NGVD. Vegetation in and along the streambed in the vicinity of the approach channel, the outlet works intake structure and the Government Canyon drainage channel will also be removed.

Hydrologic Facilities

3-18 The device used to record the water surface elevation of the Seven Oaks reservoir will be a servomanometer. This includes a purge bubbler system, a nitrogen storage tank, and a 3-inch rigid water-tight conduit installed just beneath the upstream face of the dam that runs from the top of the dam down to the reservoir bottom. Six orifices with pull boxes at 0, 20, 120, 220, 320, and 420 feet above the invert (measured vertically) will be installed in the 3-inch conduit. In addition, pull boxes will be installed in the conduit every 100 linear feet to provide access to the conduit for installation and maintenance. Six polyethylene tubes will be inserted within the conduit, one for each orifice. The two lowest orifices will be enclosed with a sediment screen/tower, 20 feet high, to protect the orifice from being buried in sediment. A strip-chart water surface recorder will be installed to automatically record water surface measurements from the servomanometer. A 8'W by 8'L x 14'H (inside height) gauge house will be constructed on the top of the dam to contain the hydrologic instrumentation and to provide AC power, if there is not a control house available for this equipment. A digital readout display will be attached to the recorder to display the latest recorded measurement.

3-19 A series of staff gauge boards will be installed along the upstream face of the dam. The boards will be graduated in 0.10 foot increments. Each staff gauge board will be at least 5 feet high and attached to a galvanized steel channel set in concrete. Each staff board should be installed so as to be readable from the top of the dam and the control house. The elevation of these staff gauges should be adjustable.

3-20 Each outlet service gate will have an automatic recorder to document all gate movements. These recorders will monitor gate settings and make a permanent record of them. They will be connected to the gate control mechanisms and the radio telemetry equipment. The recorders should be automatically activited each time a gate control switch is activated and a recording of the new gate setting with the date and time will be made. This information will be recorded on paper and transmitted to the District Office via the radio telemetry equipment.

3-21 Tipping-bucket rain gauges will be installed at the dam, at the stream gauging station downstream of the dam, and at three other locations upstream from the dam. These locations are: Big Bear Lake Dam, Camp Angeles, and Seven Oaks. The tipping bucket will accumulate .01 inch of rainfall before each tip.

3-22 The existing USGS stream gauging station "Santa Ana River near Mentone," No. 11051500, will be used to measure the outflow from the Seven Oaks Dam outlet works. This gauge has a period of record from July 1896 to the present. Staff gauges will also be installed to allow visual measurement of the water surface at the gauge. A digital readout display will be attached to the recorders for the main gauge and the supplemental gauge to display the latest recorded measurements. The gauge control section and control house may require modifications to ensure that they can continue in operation during the maximum possible controlled release from Seven Oaks Dam of 8,000 ft^3/s.

3-23 At least three index range lines will be established in the reservoir area to provide an indication of the amount of sediment deposition occurring in the reservoir during the project life and to help determine the need for additional topographic mapping.

3-24 A "remote terminal unit" (RTU) compatible with the existing Los Angeles District's telemetry system will be installed at the reservoir water surface level gauge house, the three upstream precipitation stations, and the downstream gauge house. Each RTU will include a power supply, hydrologic instrument interface, shaft interface to connect to the water surface recorder (if applicable), programmable micro-computer control, radio transceiver, and antenna. These RTU's will transmit the gauged information to the District Office via a radio repeater station. A new radio repeater will be necessary to ensure reception of the telemetry transmissions from the gauge locations described above. This repeater would be located on an accessible mountain peak location that has line-of-sight transmission capability to repeat signals from the gauge locations to the Los Angeles District Office.

Relocations

3-25 There will be no permanent relocation of any public streets, highways, or railroads. Also see section VIII. An unimproved access road used by the U.S. Forest Service and Southern California Edison Company (SCE) to maintain hydroelectric facilities in the Upper Santa Ana Canyon will be relocated further west. The SCE hydropower facilities and a water well owned by the Bear Valley Mutual Water Company (BVMWC) will also be relocated (pl. 22).

3-26 The SCE hydropower facilities consist of a series of powerhouses, flume lines, and transmission lines. The relocation plan eliminates powerhouse No. 2 and transfers the hydraulic head to powerhouse No. 3 where new, high efficiency turbines will be installed. The flume line will be relocated by replacing the existing flume with a closed, pressurized conduit along the streambed. Lastly, the transmission line will be relocated up above the spillway crest elevation, 2,850 feet NGVD, along the east rim of the reservoir.

3-27 The BVMWC Well No. 2 will be relocated downstream in the vicinity of Powerhouse No. 3. Other facilities within the canyon and borrow areas will be protected in place or avoided.

Esthetic Treatment

3-28 The construction of Seven Oaks Dam will provide a large, prominent, south-sloping downstream dam face with side slopes of 1 vertical on 2 horizontal, rising approximately 550 feet above the streambed at its centerline. The curved embankment will be about 3,000 feet in length. The spillway for Seven Oaks Dam will be excavated to the east of the dam and will be 500 feet wide and unlined.

3-29 Although the project is located in a highly scenic area, no esthetic treatment will be recommended for the spillway due to its location. The spillway will not be readily seen by the public from the proposed entry road or mouth of the canyon.

3-30 The dam structure will easily dominate the visual setting due to its proportion and scale relative to the existing characteristics of the landscape. However, it is assumed that only one third or less of the upper portion of the downstream face of the dam will be viewed from the entry road and from outside the canyon. Therefore, esthetic treatment is proposed for only the upper one third portion of the dam.

3-31 The upper one-third portion of the downstream embankment of the dam will be stained with a color similar to the boulders and rock outcroppings on the surrounding hillsides. Similar colored rock may also be obtained elsewhere. The color stain and/or rock added to the downstream rock face will blend the portions of the dam visible from outside the canyon with foreground, background, and adjacent hillsides, making the dam structure less alien to the surroundings.

Administration Area During and After Construction

3-32 During construction of the project, a small office will be built downstream from the damsite. The location will be north of Greenspot Road just off the existing SCE access road which leads into the damsite (pl. 24). The office will be used as a Corps of Engineers' resident office during construction. The office will be a 10-foot by 25-foot concrete masonry block building with an attached 2-car garage. Temporary trailers will also be used as office space during construction. Electrical power will be supplied from a distribution line extension to be built and owned by Southern California Edison (SCE) for this project. Power (208Y/120 or 120/240 volts) will be obtained from a step-down transformer installed and owned by SCE. Power will supply lighting panelboards that will feed loads such a luminares, receptacles, and other small loads. There will also be an unpaved parking lot area. After completion of the project, the building will be converted into the dam tender's office and maintenance yard for project operation.

IV. HYDROLOGY

General

4-01 This section presents a brief description of the Seven Oaks Dam drainage area along with a presentation of design discharges, water surface elevations and other pertinent information for the recommended dam. More detailed information on the development of the hydrology is given in Volume 7 of this Phase II GDM.

4-02 The Seven Oaks Dam watershed drains approximately 177 mi^2, excluding the ineffective area of 32 mi^2 tributary to Baldwin Lake. The headwaters of the watershed are in the rugged San Bernardino Mountains. Elevations vary from 10,664 feet NGVD at Anderson Peak and 11,502 feet NGVD at San Gorgonio Peak to 2,060 feet NGVD at the damsite, which is approximately 1 mile upstream from the canyon mouth. Generally trending northeast to southwest, the 27 miles of river upstream from the damsite have an average gradient of 300 ft/mi, with individual stream gradients of 450 and 628 ft/mi for subareas A2 and A3, respectively (pl. 32). However, small tributaries originating in the high mountains have gradients that exceed 1,900 ft/mi. Bear Creek, the principal tributary within the canyon area, comprises 55 mi^2 and has an average gradient of approximately 460 ft/mi. Well-developed growths of fir and pine occur above elevations of about 5,000 feet. Large areas on the plateaus and hills are covered with grass and brush. The drainage area above the dam is expected to remain largely undeveloped during the project life. Big Bear Lake Dam is the only existing structure which would affect floodflows in this watershed. Big Bear Lake is a water-conservation and recreation reservoir owned by the Big Bear Muncipal Water District. The lake has a drainage area of about 38 mi^2 and has a surcharge storage of about 8,600 acre-feet at the top of the dam.

4-03 Precipitation characteristically occurs in the form of rainfall, although in the higher elevations some falls as snow. In general, the quantity of precipitation increases with elevation. The mean seasonal precipitation for the basin varies from about 24 inches at the damsite to about 45 inches in the higher mountain areas. Nearly all precipitation occurs during the months of December through March. Rainless periods of several months during the summer are common.

4-04 Streamflow is perennial in the upper canyons of the Santa Ana River and in the headwaters of most of the tributaries, but intermittent at the damsite and the river is usually dry in the summer months. Streamflow increases rapidly in response to effective precipitation. High-intensity precipitation in combination with the effects of steep gradients and possible denudation by fire result in intense sediment-laden floods, with some debris in the form of shrubs, trees and boulders.

Design Criteria

4-05 The recommended plan is to construct a 550-foot high, earth and rock fill dam about 1 mile upstream from the confluence of Mill Creek and the Santa Ana River. The top of the dam is at elevation 2,610 feet NGVD (National Geodetic Vertical Datum) and the spillway crest tops out at elevation 2,580 feet NGVD. Total storage below spillway crest of about 145,600 acre-feet is based on NED (National Economic Development) considerations. Of the 145,600 acre-feet of storage, 32,000 acre-feet is set aside for 100-year sediment accumulation, which is the total amount of sediment trapped in the reservoir over a 100-year period for sediment producing storms in that period. The operation of Seven Oaks Dam during major flood events is based on the flood condition downstream at Prado Dam. When the water surface in Prado Dam is rising, a maximum release of 500 ft^3/s will be maintained from Seven Oaks Dam. As the water surface elevation at Prado Dam begins to decline, a schedule maximum discharge of up to 7,000 ft^3/s will be released (under emergency conditions, 8,000 ft^3/s). With the reservoir design flood (RDF) occurring at Prado Dam and releases at Seven Oaks Dam held to a maximum of 500 ft^3/s, less than 4,000 acre-feet of the Seven Oaks Dam component 4-day volume of 101,000 acre-feet will reach Prado Dam. When the water surface elevation at Prado Dam is falling, the release rate at Seven Oaks Dam is based on the water surface elevation behind Seven Oaks Dam. The standard project flood (SPF) with a peak inflow of 82,000 ft^3/s and a 4-day volume of 110,500 acre-feet (333-year frequency event) fills the reservoir to a maximum water surface elevation of 2574.93 feet NGVD or 5.07 feet below spillway crest. The RDF with an estimated peak discharge of 85,000 ft^3/s and a 4-day volume of 115,000 acre-feet, corresponds to a 350-year frequency event and will fill the reservoir to the spillway crest. The initial water surface elevation for the SPF and RDF routings is elevation 2,300 feet NGVD (top of debris pool) for future conditions. The appropriateness for this elevation is manifested in the water control plan for Seven Oaks Dam. The water control plan requires the release of floodflows as soon as possible to draw down the reservoir pool to the top of the debris pool in preparation for the next flood event. The RDF routing is shown on figure 1. Plate 33 displays design flood peak discharges for the Santa Ana River.

4-06 Risk is defined as "the probability that one or more events will exceed a given flood magnitude within a specified period of years." In other words, risk enables a probabilistic statement to be made about the chances of a particular location being flooded when it is occupied for a

specified number of consecutive years. For example, if a project is designed to protect against a 100-year flood, during any 100-year period, there is a 63 percent risk that one or more floods will occur that exceeds the design level. The following is the results of the risk analysis at Seven Oaks Dam:

Period of Time (years)	Project Design Level Annual Exceedence Frequency (percent)	Risk of Exceeding Design Level (percent)	
		One or More Floods	Two or More Floods
10	approximately 0.29	3	0
25	approximately 0.29	7	0
50	approximately 0.29	14	1
100 (project life)	approximately 0.29	25	3

From the results listed above for the 100-year project life, a 25 percent risk exists that one or more floods will exceed the project design level of about 350 years. To illustrate, if 100 projects were designed to protect against a 350-year flood during any 100-year period, 25 of the projects will experience one or more floods exceeding the design level.

V. HYDRAULIC DESIGN

General

5-01 The hydraulic design and analysis documented in this chapter was performed in accordance with the guidance presented in EM 1110-2-1603 (ref. 2), EM 1110-2-1614 (ref. 6), EM 1110-2-2300 (ref. 9), EM 1110-2-1601 (ref. 17), and EM 1110-2-1602 (ref. 19). The references used herein are cited at the end of this chapter. A complete hydraulic design has been prepared for three alternative outlet works configurations: upstream control, downstream control, and mid-tunnel control. The hydraulic design of the plunge pool and the diversion of water during construction is dependent upon the outlet works configuration; consequently, a separate design was prepared for each alternative. However, the area-capacity curve, spillway, dam freeboard, minimum revetment design, and downstream floodway are common to all the outlet works alternatives.

Reservoir Area and Capacity

GROSS CAPACITY

5-02 The elevation versus gross storage curves are displayed in figure 2 and are based upon topographic mapping compiled from aerial photography flown in October, 1986. The mapping is on file in the Los Angeles District Office under file number CA 2777.

SEDIMENTATION

Volume

5-03 The volume of sediment to be distributed in the Seven Oaks Reservoir was determined, in reference 16, to be 32,000 acre-feet. This volume represents the amount of sediment which would be trapped in the reservoir after 100 years of service.

Selection of Method of Distribution

5-04 The procedure used was that recommended in EM 1110-2-4000 Appendix E, (ref. 1). EM 1110-2-4000 describes four methods for distributing sediment in reservoirs. Of the four, two, the "Empirical Area Reduction Method" and the "Area-Increment Method," are recommended for use.

5-05 The Empirical Area-Reduction Method is considered more accurate in predicting bed elevation at and near the dam. It is recommended for use under extreme reservoir operation conditions, when the reservoir shape is unusual, or when the 100-year volume of sediment to be distributed exceeds fifteen percent of the total reservoir capacity.

5-06 EM 1110-2-4000 does not provide a definition of "extreme operating conditions." However, in comparison with many dams which have large conservation pools, any single purpose flood control reservoir which is nearly dry at the beginning of an event, then fills rapidly and then is drawn down until it is nearly dry again as quickly as possible, could probably be characterized as having extreme operating conditions.

5-07 EM 1110-2-4000 does not provide a definition of an "unusual reservoir shape"; therefore, there is no basis for classifying the Seven Oaks reservoir as either usual or unusual.

5-08 Seven Oaks Dam will have a gross storage capacity of 145,600 acre-feet at spillway crest. Thirty-two thousand acre-feet of sediment storage is 22 percent of 145,600 acre-feet. This percentage of reservoir capacity taken up by sediment exceeds the range of applicability of the Area-Increment Method. Based upon this and the discussion in paragraphs 5-04 to 5-06, the Empirical Area Reduction Method was selected to distribute sediment in the Seven Oaks Reservoir.

Tasks in Distributing Sediment

5-09 The first task in distributing the sediment is to determine the elevation of the spillway crest. Based upon an economic analysis, elevation 2,580.0 feet NGVD was selected.

5-10 The second task is to classify the reservoir as either: Type I (Lake), Type II (Floodplain-foothill), Type III (Hill) or Type IV (Gorge). The higher the classification, the more the sediment has a tendency to deposit near the embankment. The basis of the classification is the inverse of the slope of a line plotting the logarithm of the depth on the y-axis against the logarithm of the capacity on the x-axis. EM 1110-2-4000 assigns the variable "m" to this quantity. Seven Oaks Dam has an average "m" of 2.43, which places it in the Type III (Hill) classification. Since this value is near the boundary of the Type II (Floodplain-foothill) classification, a case might be made for reducing the classification of the Seven Oaks reservoir; however, EM 1110-2-4000 indicates that reservoirs with operation schedules which require substantial drawdowns would tend to have higher classifications than that indicated by the "m" value. Therefore, it was determined that a lower classification would not be appropriate and that the Type III classification would be more accurate.

5-11 The third task is to determine the depth of sediment which will deposit against the reservoir embankment. EM 1110-2-4000 describes and provides an example of an empirical procedure for determining this value. This procedure was followed for Seven Oaks Dam and indicated that approximately 165 feet of sediment will deposit against the dam embankment and intake tower.

5-12 The fourth and final task is to distribute the sediment throughout the rest of the reservoir. EM 1110-2-4000 describes and provides an example of an empirical method for this determination also. This procedure was followed for Seven Oaks Dam. Refer to figure 2 for the resultant net area-capacity curve.

Debris Pool

5-13 As indicated in para. 8-04 of Volume 7, Hydrology, a debris pool will be maintained during the flood season at elevation 2,200 feet NGVD at the beginning of the project life, rising gradually to 2,300 feet at the end of the project life. The purpose of the debris pool is to provide a zone of still water to induce deposition of suspended sands. Deposition of this material will reduce wear on the outlet works conduits and appurtenances caused by abrasion of the high velocity flows. As shown on plate 29, the debris pool at the beginning of the project life will daylight against the existing streambed approximately 3,500 feet upstream of the toe of the dam. At the end of the project life, the debris pool will daylight against the estimated sediment delta approximately 2,000 feet upstream of the toe of the dam. These distances will be sufficient to minimize transport of sands through the debris pool to the outlet works.

Outlet Works

5-14 A complete description of the three outlet works tunnel control alternatives from the intake tower to the end of the outlet channel is given in Appendix B.

Plunge Pool for Upstream Control Alternative

5-15 The hydraulic design of the plunge pool for the upstream control alternative is discussed in detail in Appendix G. Refinement of the design with additional analysis and hydraulic model testing will be made during preparation of the Feature Design Memorandum. The plunge pool is displayed on plate 10A.

Flip Bucket and Plunge Pool for Downstream Control Alternative

GENERAL

5-16 The development of conceptual alternatives for dissipating the energy of releases from the Seven Oaks Dam outlet works and for the preliminary evaluation of those alternatives is discussed in Appendix G.

FINAL ENERGY DISSIPATING ALTERNATIVES

5-17 Completion of the preliminary evaluation discussed in Appendix G left the following two conceptual alternatives to be given more detailed consideration:

a. Alternative 2, pre-formed plunge pool and toe protection (cutoff wall) for the outlet exit channel.

b. Alternative 7, flip bucket with pre-formed plunge pool and toe protection (cutoff wall) for the outlet exit channel.

5-18 These alternatives are very similar. Both consist of a cutoff wall and a pre-formed plunge pool. The main difference between the two is that Alternative 7 has a flip bucket which will give discharges coming from the exit channel a longer trajectory. With the flip bucket the discharges will impact the streambed further from the end of the exit channel, and the cutoff wall will not have to be as deep. On the other hand, because the flip bucket tends to spread the various discharges over a larger area, the scour hole (and therefore the preformed plunge pool) will be larger.

5-19 The procedure used in the final analysis of the alternatives was as follows:

a. Design the flip bucket.

b. Estimate trajectory lengths for the various design discharges for both alternatives.

c. Estimate the scour hole depths for the various design dicharges and trajectories for both alternatives.

d. Estimate the scour hole and pre-formed plunge pool dimensions for both alternatives.

e. Estimate the cutoff wall depths for both alternatives.

f. Estimate the excavation volumes for both alternatives.

g. Estimate the concrete volumes for both alternatives.

h. Estimate the cost of each alternative.

i. Select the alternative.

j. Estimate the misoperation scour hole for the selected alternative for dam safety purposes.

DESIGN DISCHARGES

5-20 Both alternatives were designed to function under a range of discharges varying from 500 to 8,000 ft^3/s. For each design discharge there were two sets of velocities and depths representing conditions at the end of the exit channel (sta. 30+40). One set assumed a smooth outlet works conduit to estimate the maximum possible velocities, and the other set assumed a rough outlet works conduit to estimate the maximum possible depth. Where possible, balanced gate operation was assumed. In the case of the maximum depth set, a 50 percent bulking due to air entrainment was assumed. Finally, in order to analyze the effect of misoperation, two sets of discharges, depths and velocities were used. These sets represent the conditions that would result if both of the main operating gates were wide open with the reservoir full to spillway crest for the maximum velocity and the maximum depth assumptions. The design and misoperation discharges, velocities and depths are as follows:

Table V-1. Hydraulic Parameters for Flip Bucket and Plunge Pool Analysis and Design for Downstream Control Alternative.

Condition	Discharge (ft^3/s)	Velocity (ft/s)	Depth (ft)
Max. Velocity	500	80	0.57
Max. Velocity	2000	115	0.79
Max. Velocity	4000	142	1.30
Max. Velocity	8000	145	2.50
Max. Velocity	11440*	120	4.30
Max. Depth	500	31.6	2.10
Max. Depth	2000	55	2.60
Max. Depth	4000	65.6	4.20
Max. Depth	8000	79	6.90
Max. Depth	8614*	69	8.60

*misoperation discharge

FLIP BUCKET DESIGN

Radius of Curvature

5-21 The flip bucket was designed for the 8,000 ft^3/s maximum design discharge and was checked for both the maximum velocity and maximum depth condition. The minimum radius of the flip bucket was based upon the following equation:

$$r_{min} = \frac{\text{rho } V_1^2 \, d_1}{P_t - \text{gamma } d_1}$$

where:

r_{min} is the minimum bucket radius

rho is the mass density of water

gamma is the unit weight of water

d_1 is the depth of water at the initial point of curvature

P_t is the allowable bucket pressure, and

V_1 is the velocity at the initial point of curvature.

This equation was provided in reference 20. Using this equation a bucket radius of curvature of 120 feet was found to satisfy both the maximum velocity and maximum depth condition.

Minimum Deflection Angle

5-22 The minimum deflection angle required to turn the jet on the flip bucket was estimated by the following equation:

$$\text{theta} = \text{Tan}^{-1}((d_1 \ (2r-d_1)).5/(r-d_1))$$

where:

r is the bucket radius

d_1 is the depth of water at the initial point of curvature, and theta is the minimum deflection angle

The source of this equation is reference 20. The minimum deflection angle was determined to be 19.5 degrees. For this design a deflection angle of 35 degrees was selected. This is the maximum deflection angle recommended in reference 21.

Bucket Height

5-23 The following equation was used to estimate the flip bucket height:

$$h = r - r \cos(\text{theta})$$

where:

h is the bucket height

r is the bucket radius, and

theta is the deflection angle

The bucket height was determined to be 22.7 feet.

TRAJECTORIES

Throw Distance

5-24 The throw distance was determined graphically by plotting the trajectory of the discharges on a sheet with the streambed profile.

Impact Angle

5-25 The angle of impact of the discharges with the streambed was estimated using the following equation:

$$\text{theta}' = \text{Tan}^{-1} (\text{Sec}(\text{theta}) (\text{Sin}^2(\text{theta}) + Y_1/h_e)^{.5})$$

where:

theta' is the angle of impact

theta is the deflection angle, and

Y_1 is the vertical distance between the edge of the flip bucket or exit channel and the point of impact.

SCOUR HOLE

Jet Dispersion

5-26 Figure 157 of Reference 21 was used to estimate the dispersion of the jet exiting the flip bucket. This figure estimates angles of lateral dispersion based upon the Froude Number for flip bucket deflection angles varying between 15 and 35 degrees. For the alternative without the flip bucket no dispersion was assumed. It was also assumed that there would be no vertical dispersion for either alternative. The jet dispersion computations were used to estimate the velocity of the jet at impact with the streambed for use in the scour equation.

Scour Hole Depth

5-27 The scour hole depths were computed for the full range of design discharges for both alternatives using two equations published in reference 22. The equations are as follows:

$$H_v = 0.558\ V_i\ d_i\ /\ (d_m^{0.1})$$

where:

H_v is the vertical jet scour depth (meters)

V_i is the impact velocity in m/s

d_i is the flow depth at impact (meters)

d_m is the mean particle diameter (meters)

Angled Jet Depth = H_v Sin (theta') - d_{tw}

where:

theta' is the impact angle

d_{tw} is the tailwater depth

The mean particle diameter was assumed to be 15 mm. This is considerably finer than the mean diameter of the material presently exposed on the surface of the streambed; therefore, it is considered to be a conservative assumption. Additionally, the tailwater depth was assumed to be zero, which is also conservative.

Scour Hole Configuration

5-28 The scour hole that would result from any given discharge was assumed to have bottom dimensions as follows. The width of the scour hole would be four times the width of the jet as it impacts the streambed. The length of the scour hole would be four times the depth of the scour hole. The sides of the scour hole were assumed to slope at 3 horizontal to 1 vertical.

5-29 For each alternative, 8 scour holes were computed. For each of the four design discharges, a scour hole was computed for both the maximum velocity and the maximum depth conditions. The 8 scour holes were plotted on both plan and profile. The ultimate scour hole that will develop for the entire range of normal operating conditions was estimated by graphically determining a single scour hole that encompasses all of the 8 individual scour holes. The following tables provide the estimated ultimate scour hole dimensions for each alternative.

Table V-2. Dimensions of Ultimate Scour Hole With Flip Bucket for Downstream Control Alternative.

Station	Approximate Depth (ft)	Bottom Width (ft)
30+54	0	119
30+62	5	119
30+85	5	119
31+51	11	152
31+91	15	172
32+49	20	201
34+70	13	217
37+10	21	284
37+96	21	284
38+45	0	284

Tabel V-3. Dimensions of Ultimate Scour Hole Without Flip Bucket for Downstream Control Alternative.

Station	Approximate Depth (ft)	Bottom Width (ft)
30+54	0	96
31+00	37	96
31+26	43	96
32+97	37	96
33+97	0	96

PRE-FORMED PLUNGE POOL

Dimensions

5-30 The pre-formed plunge pool for each alternative was assumed to be two thirds the depth of the ultimate scour hole. The following tables provide the dimensions of the pre-formed plunge pools.

Table V-4. Dimensions of Pre-formed Plunge Pool With Flip Bucket for Downstream Control Alternative.

Station	Approximate Depth (ft)	Bottom Width (ft)
30+54	0	119
30+62	3.3	129
30+85	3.3	129
31+51	7.3	174
31+91	10.0	202
32+49	13.3	241
34+70	8.7	243
37+10	14.0	326
37+96	14.0	326
38+45	0	284

Table V-5. Dimensions of Pre-formed Plunge Pool Without Flip Bucket for Downstream Control Alternative.

Station	Approximate Depth (ft)	Bottom Width (ft)
30+54	0	96
31+00	24.7	170
31+26	28.7	182
32+97	24.3	169
33+97	0	96

Excavation Volumes

5-31 The required excavation volumes for the pre-formed plunge pools were computed by the average end area method. With the flip bucket an excavation volume of 91,305 yds^3 would be required. Without the flip bucket an excavation volume of 65,605 yds^3 would be required.

CUTOFF WALLS

Depths

5-32 The cutoff walls were assumed to be 10 feet deeper and 10 feet wider on each side than the scour hole dimensions where they come into contact with the cutoff walls. The cutoff walls were assumed to have a 1H on 1V sideslope extending below ground with 15-foot-long tie back walls at each end. The cutoff walls were assumed to be 1.5 feet thick. For the alternative with the flip bucket, the elevation of the toe of the cutoff wall is 2,018 feet NGVD. For the alternative without the flip bucket, the elevation of the toe of the cutoff wall is 1,981 feet NGVD.

Concrete Volumes

5-33 The volume of concrete in the flip bucket is approximately 580 yds^3. The volume of concrete in the cutoff walls is approximately 370 and 1,900 yds^3, respectively, for the with and without flip bucket alternatives.

SELECTION OF ENERGY DISSIPATING ALTERNATIVE

5-34 Based on a preliminary cost comparison between the with flip bucket alternative and the without flip bucket alternative, in conjunction with their respective designs on the pre-formed plunge pool and cutoff wall, the with flip bucket alternative is selected as the more economical plan.

MISOPERATION

5-35 Using the methods described above, the ultimate scour hole resulting from the misoperation discharges with the selected energy dissipating plan was computed. The closest approach of the daylight line of the misoperation scour hole to the toe of the main dam embankment is approximately 340 feet, which is considered a reasonably safe distance. Since misoperation is by definition an unintentional deviation from normal operation, the misoperation scour hole was not used in the recommended design.

SELECTED ENERGY DISSIPATING PLAN

General

5-36 The recommended design is Alternative 7, the flip bucket with pre-formed plunge pool and toe protection (cutoff wall) for the outlet channel. For this alternative, normal operation releases would discharge from the flip bucket. Under these conditions a scour hole would be formed by the resulting water jet, eventually providing the required energy dissipation. By pre-forming a plunge pool downstream from the flip bucket, energy dissipation will be achieved for first-time releases. Therefore, it is recommended that a plunge pool be pre-formed to approximately two-thirds the ultimate scour depth. Without providing toe protection for the outlet exit channel, the channel would eventually be undercut by the scour hole induced by low flows. Therefore, it is further recommended that the existing ground below the outlet portal exit channel be protected with a 1V on 1H sloping cutoff wall. Some minor grading is required downstream from the plunge pool to provide a smooth transition to the existing channel.

Pre-formed Plunge Pool

5-37 The plunge pool has a total top length of approximately 790 feet. The top width varies between approximately 130 and 460 feet. The maximum depth of the pre-formed pool is approximately 14 feet. It is suggested that when excavating the plunge pool, the coarse surface material (diameter of 6 inches) be retained and used to line the plunge pool after excavation is complete. This will provide additional stability for the plunge pool and may reduce the ultimate scour depth. It is further suggested that surface material, with diameter 12 to 18 inches, be used to line the upstream face of the preformed plunge pool. This will provide additional protection against scour in the vicinity of the cutoff wall.

Cutoff Wall

5-38 Toe protection for the flip bucket will be provided by a concrete cutoff wall. This wall will be approximately 24 feet wide just under the flip bucket (elevation 2,050 feet NGVD). The wall extends down at a 1V on 1H sideslope and widens to 169 feet at elevation 2,033 feet NGVD. The wall then continues down at this width to elevation 2,018 feet NGVD. For purposes of producing a cost estimate, the wall was assumed to be 1.5 feet thick. Structural considerations may dictate otherwise. The cutoff wall will have tie back walls approximately 15 feet long from elevation 2,033 to elevation 2,018. Once the scour hole is fully developed, all but the lower 10 feet of the cutoff wall will be exposed. Therefore, the cutoff wall must be anchored to prevent failure by sliding.

Plunge Pool for Downstream Control Alternative

MODEL STUDY

5-39 The Chee and Yuen method was used for computing scour depths. It must be recognized that considerable uncertainty exists with any currently available method for computing scour depths. Consequently, refinement of the design of both the flip bucket and pre-formed plunge pool will be made with additional analysis and hydraulic model testing during preparation of the Feature Design Memorandum.

PROJECT IMPACTS

5-40 The following impacts will be caused by the selected energy dissipating plan.

a. The proposed Seven Oaks Dam embankment will be protected against damage due to scour.

b. The proposed outlet portal exit channel will be protected against damage due to scour.

c. A scour hole (plunge pool) with dimensions described previously will be formed by normal operation reservoir releases.

d. Adequate energy dissipation will be provided by the plunge pool such that the downstream channel bed will be relatively stable.

e. The potential for slope failure of the canyon walls will be reduced.

f. The potential for scour damage to all structures, pipelines, and other facilities in the Santa Ana Canyon will be reduced or eliminated.

g. Post-project flooding downstream from the damsite will be reduced relative to pre-project flooding because post-project releases will be substantially smaller than pre-project discharges.

h. The significant armoring potential of the river channel between the damsite and Greenspot Road, and the reduction in flows downstream from the damsite will result in reduced scour potential for post-project conditions.

Flip Bucket and Plunge Pool for Mid-Tunnel Control Alternative

5-41 The hydraulic design of the flip bucket and pre-formed plunge pool for the mid-tunnel control alternative was based on the same criteria and procedures used for the downstream control alternative, as discussed in detail in paragraphs 5-16 through 5-40 above. As for the downstream

control alternative, the combination of a pre-formed plunge pool with a flip bucket was determined to be less expensive than a pre-formed plunge pool without a flip bucket.

5-42 The hydraulic parameters for the analysis and design of the flip bucket and plunge pool for the mid-tunnel control alternative are given in table V-6 below. The depths for both maximum velocity and maximum depth include an increase of 50 percent over computed values to account for air bulking. The bucket flip angle was set at 35^{o} with respect to the horizontal. The bucket radius is 57 feet and the bucket height is 10.5 feet.

Table V-6. Hydraulic Parameters for Flip Bucket and Plunge Pool Analysis and Design for Mid-Tunnel Control Alternative.

Condition	Discharge (ft^3/s)	Velocity (ft/s)	Depth (ft)
Maximum Velocity	500	28.3	1.47
Maximum Velocity	2,000	59.3	2.81
Maximum Velocity	4,000	88.2	3.78
Maximum Velocity	8,000	117.1	5.70
Maximum Velocity	12,430*	128.2	9.39
Maximum Depth	500	18.9	2.21
Maximum Depth	2,000	31.0	5.37
Maximum Depth	4,000	42.6	7.82
Maximum Depth	8,000	58.7	11.34
Maximum Depth	14,455*	66.3	15.6

*misoperation discharge

5-43 Dimensions of the ultimate scour hole and pre-formed plunge pool for both with and without a flip bucket are given in tables V-7 through V-10 below.

Table V-7. Dimensions of Ultimate Scour Hole With Flip Bucket for Mid-Tunnel Control Alternative.

Station	Approximate Depth (ft)	Bottom Width (ft)
30+55	0	0
30+65	7	50
30+80	15	100
31+15	19	116
31+60	24	144
33+30	19	192
34+65	14	252
35+40	9	252
35+70	0	0

Table V-8. Dimensions of Ultimate Scour Hole Without Flip Bucket for Mid-Tunnel Control Alternative.

Station	Approximate Depth (ft)	Bottom Width (ft)
30+55	0	72
30+60	6	72
30+65	10	72
30+70	16	72
30+80	22	72
30+85	23	72
30+90	29	72
30+95	33	72
31+05	44	72
31+55	40	72
32+90	36	72
33+50	19	72
34+05	0	72

Table V-9. Dimensions of Pre-formed Plunge Pool With Flip Bucket for Mid-Tunnel Control Alternative.

Station	Approximate Depth (ft)	Bottom Width (ft)
30+55	0	0
30+65	5	64
30+80	10	130
31+15	13	154
31+60	16	192
33+30	13	230
34+65	9	280
35+40	6	270
35+70	0	0

Table V-10. Dimensions of Pre-formed Plunge Pool Without Flip Bucket for Mid-Tunnel Control Alternative.

Station	Approximate Depth (ft)	Bottom Width (ft)
30+55	0	72
30+60	4	84
30+65	7	92
30+70	11	104
30+80	15	116
30+85	15	118
30+90	19	130
30+95	22	138
31+05	29	160
31+55	27	152
32+90	24	144
33+50	13	110
34+05	0	0

5-44 The elevation of the toe of the cutoff wall with the flip bucket is 2009 ft NGVD, and without the flip bucket the toe elevation is 1964 ft NGVD. Refinement of the design of both the flip bucket and plunge pool will be made with additional analysis and hydraulic model testing during preparation of the Feature Design Memorandum.

Spillway

5-45 Preliminary rating curves of spillway width versus embankment height were developed for three different types of rock cut emergency spillways: (1) broadcrested with sloping upstream and downstream channels, (2) entirely horizontal, and (3) horizontal with an ogee crest at the downstream end. Economic analysis determined that type (1) was the most feasible.

NEED FOR CONCRETE LINING

Hydraulic Conditions

5-46 To aid in assessing the need for lining the spillway with concrete, the hydraulic conditions at the proposed Seven Oaks spillway site were compared to those at other similar spillways for Los Angeles District dams. A list was compiled of similar existing rock cut emergency spillways designed and constructed by the Los Angeles District. The dimensions and design discharges for each of these spillways were also obtained and the channel velocities likely to be experienced under design conditions at each of these sites were estimated by using normal and critical depth relations. The results of the evaluation are as follows:

Dam	Year Constructed	Des. Q ft^3/s	Vc fps	Vn fps
Pine Canyon	1957	31,700	14.5	17.9
Painted Rock	1959	405,000	27.4	40.2
Whitlow Ranch	1960	147,000	23.3	25.5
Alamo	1968	49,000	23.1	30.5
Dreamy Draw	1973	7,000	12.3	16.3
Cave Buttes	1979	100,600	18.4	26.7
Adobe	1982	9,620	18.6	22.5
New River	1984	29,850	21.9	30.4
Seven Oaks	proposed	174,000	21.7	29.2

After reviewing this information, it was determined that the hydraulic conditions at Seven Oaks are comparable to those at the other unlined rock cut emergency spillways and that the spillway at Seven Oaks Dam could therefore remain unlined.

Other Considerations

5-47 Based upon an economic analysis, the spillway crest at the Seven Oaks site was set well above the SPF pool maximum water surface elevation. The hydrologic characteristics of the watershed above the Seven Oaks site, and the capacities of the reservoir and spillway are such that prolonged discharges over the spillway are very unlikely. In addition, geotechnical considerations and evaluations show that an unlined spillway is feasible, as explained in paragraph 2-34 and in Appendix A.

Conclusions

5-48 Because of the infrequency of use, the short duration of spillway flows, and the similarity in terms of discharge and velocity with other unlined spillways previously constructed by the Los Angeles District, it appears that an unlined spillway would not pose an excessive risk of loss of spillway control during a single event. (It is expected, however, that should the spillway ever be used there will be some damages necessitating repairs). In order to further minimize the risk of losing control of spillway discharges, two additional measures will be incorporated into the spillway design. First, a reinforced concrete sill will be constructed at the spillway crest. Should a headcut reach part of the crest, the sill should be structurally sound enough to support an undercut section. It is considered unlikely that the entire crest would be undercut. Second, the spillway crest will be located as far upstream as possible in order to maximize the length of the channel downstream from the crest. A headcut would therefore take longer to reach the control section. During the preparation of the Embankment and Spillway Feature Design Memorandum, additional studies will be conducted to assess the cost of additional erosion protection for the spillway versus the cost of probable repairs required over the life of the project.

SPILLWAY FEATURES

Crest Elevation

5-49 Based upon an economic analysis, elevation 2,580.0 feet NGVD was selected as the spillway crest elevation.

Alignment, Width and Side Slopes

5-50 The alignment of the spillway centerline is shown on plate 7. The spillway base width was determined to be 500 feet based upon economic analysis. The cut side slopes were determined to be 1H on 2V with benches based upon geotechnical considerations. The overall slope averaged 1V on 1H.

Approach Channel

Slope

5-51 The slope of the approach channel to the spillway crest was set at -0.025 (adverse). This is based upon information provided in reference 3. This reference is the theoretical basis for previous spillway slope design in the Los Angeles District. An adverse grade of -0.025 leading up to the spillway crest is provided to avoid the formation of waves on the spillway crest.

Length

5-52 The length of the approach channel was set at approximately 235 feet. This was set so that the ratio of head to approach channel length would be the same in the proposed spillway as it was in the experimental models used in reference 3. This approach channel length meets the criteria in EM 1110-2-1603 (ref. 2). Paragraph 41 of EM 1110-2-1603 indicates that "any such control structure or break in grade should be far enough downstream so that the sideslopes of the cut are sufficiently high to confine the flow at maximum reservoir level for design flow." The sides of the cut at the location of the proposed spillway crest daylight at approximately elevation 2,800 feet NGVD, while the maximum reservoir water surface is at elevation 2,604.4 feet NGVD. Since 2,800 feet NGVD is much higher than 2,604.4 feet NGVD, the flow will therefore be well confined.

SPILLWAY DESIGN

Approach Channel Roughness

5-53 A Manning's roughness coefficient of 0.035 was selected for the spillway channel upstream from the crest. This was based on the average value for smooth and uniform rock channels in table IV, paragraph 19.a. of EM 1110-2-1603.

Spillway Rating Curve

5-54 The HEC-2 Water Surface Profiles Computer Program was used to compute the water surface in the reservoir for a range of spillway discharges. Cross sections for the HEC-2 model were taken at intervals varying from 25 to 200 feet apart, extending from the spillway crest to a point in the reservoir about 275 feet past the upstream end of the spillway cut. Critical depth was assumed for the starting downstream water surface elevation. The spillway rating curve is displayed in figure 3.

Design Discharge

5-55 A reservoir routing was performed based upon the spillway rating curve. It was determined that the spillway design flood would produce a maximum water surface elevation of 2604.4 feet NGVD and a maximum discharge of 174,000 ft^3/s.

Downstream Channel Roughness

5-56 The Manning's roughness coefficients in the spillway channel downstream from the spillway crest were determined by a trial and error procedure using equations 31 and 32 in paragraph 43.d. of EM 1110-2-1603. An overbreak (roughness element height) of 0.5 feet was assumed for the channel. The resulting roughness coefficients varied from 0.027 at the spillway crest to 0.031 at the downstream end of the spillway for the design discharge.

Downstream Channel Slope

5-57 General practice in a channel downstream from a spillway crest designed for critical depth control is to set the channel slope greater than the critical slope in order to maintain supercritical flow. This avoids the possibility of a backwater drowning out the control at the spillway crest. Unfortunately, the critical slope varies widely with discharge, especially when adjusting the roughness coefficient in the channel using the equations mentioned in the previous paragraph. It would be impossible to provide a supercritical slope for all discharges. For example, for the design discharge of 174,000 ft^3/s the critical slope is approximately 0.0044, but for a discharge of 3,000 ft^3/s the critical slope is about 0.15. In order to determine a reasonable compromise slope, the designs of existing similar rock cut spillways in the Los Angeles District were reviewed. There are nine such spillways with channel slopes downstream from the spillway crests varying from 0.010 to 0.029. Four of the nine have slopes of 0.020. A channel slope of 0.020 was therefore selected as a reasonable compromise. At a slope of 0.020 in the channel downstream from the spillway crest, discharges greater than 20,000 ft^3/s will be in the supercritical flow regime and control will be at the crest. Only for discharges less than 20,000 ft^3/s, which is a small fraction of the design discharge of 174,000 ft^3/s, will control at the crest be lost.

Channel Velocities

5-58 A supercritical HEC-2 model was developed to estimate the water surface profiles and flow velocities in the channel downstream from the spillway crest. Cross sections were taken at intervals varying from 25 to 100 feet. The HEC-2 model developed for the spillway rating curves was used to estimate the design water surface profiles and flow velocities upstream from the spillway crest. The flow velocities in the spillway channel for the design discharge of 174,000 ft^3/s vary from 12.4 feet per second at the upstream end, to 22.4 ft/s over the crest, to 32.3 ft/s at the downstream end.

Profiles

5-59 Refer to plate 8 for profiles showing the proposed spillway invert slope, design water surface, energy grade line and flow velocities.

Freeboard

FREEBOARD REFERENCE LEVEL

5-60 Based on paragraph 2c. of reference 4., the final top of dam elevation at Seven Oaks Dam will be selected using one of the following three criteria:

a. Freeboard Reference Level (FRL) B plus 3.0 feet. FRL B is the highest water surface elevation produced by routing the spillway design flood (in this case the PMF) with the starting water surface elevation set at the spillway crest.

b. FRL B plus the wave runup plus the wind setup. Reference 4. refers (indirectly) to EC 1110-2-27 (ref. 13) for further guidance. Paragraph 12f and enclosure 5 of EC 1110-2-27 indicate that the "significant" wave runup should be used.

c. FRL C plus 5.0 feet. FRL C is the highest water surface elevation produced by routing the spillway design flood with the starting water surface elevation set at 50 percent of the flood control pool.

Reservoir routings were performed to determine FRL B and FRL C. Both were determined to be elevation 2,604.4 feet NGVD.

WAVE RUNUP

5-61 In order to determine wave runup, the design significant wave height must first be determined. From reference 4., the method presented in ETL 1110-2-305 (ref. 5) was used for this determination. The procedure was applied as indicated in the following paragraphs.

a. First, wind roses were developed to obtain the maximum one-hour overland winds 33 feet above the ground, centered at 0, 12, 24, 48, and 72 hours after the maximum precipitation.

b. Second, design fetch lengths were determined for the east, east-northeast, northeast, north-northeast, and north directions. Figure V-3 from the Phase I GDM Supplement, Main Report was used as the basis for the fetch lengths. The design fetch length was taken as the arithmetic average of nine fetches measured over a 24 degree sector centered around the nominal wind direction. The fetches were measured from the top of the dam embankment to the end of the maximum reservoir pool. The location on the top of the dam embankment from which the fetches were measured was located by visual estimate such that the average fetch length would be maximized.

c. Third, based upon examination of Plates B-14 and C-7 in the Phase I GDM Supplement, Appendixes, it was determined that the maximum reservoir water surface elevation at Seven Oaks Dam occurs approximately three hours after the maximum precipitation. The maximum one-hour overland wind centered three hours after maximum precipitation was determined for each fetch direction by plotting the maximum one-hour overland winds centered at 0, 12 and 24 hours after maximum precipitation and interpolating graphically with the aid of a french curve.

d. To obtain the fastest mile, and maximum 2-hour, 3-hour, 4-hour and 6-hour winds centered three hours after maximum precipitation, the maximum 1-hour wind was multiplied by 1.25, 0.96, 0.93, 0.91 and 0.88 respectively.

e. To convert to overwater wind speeds, all the overland wind speeds were multiplied by 1.1. None of the fetch lengths were longer than 10 miles; therefore, no attempt was made to compute a wind stability correction factor based upon air and water temperatures.

f. Figure 4 of ETL 1110-2-305 was used to plot fetch limited and duration limited curves of wave height versus wind duration. From the intersection of the fetch and duration limited wave height versus duration curves for each fetch direction, the design deepwater significant wave height for each direction was determined.

g. Because the duration of the design deepwater wave was less than 30 minutes for all the fetches and because most of the fetches were less than one mile long, extensive extrapolation was necessary when using figure 4 of ETL 1110-2-305. This caused some concern about the accuracy of the results.

h. In order to check the results, the method for determining deepwater wave heights presented in the 1984 edition of the Shore Protection Manual (SPM) was used (ref. 7). Since it appears that Figure 4 in ETL 1110-2-305 is based upon this method, the results should be comparable, and cross checking results where the methods overlapped indicated that they were. The method presented in the SPM provides better definition of deepwater wave heights for the fetch limited curves for small fetches because it is based upon equations rather than a nomograph. The SPM method also provides better definition of deepwater wave heights for the duration limited curves for short durations because it extends to 20 minute rather than 30 minute durations.

i. The results of both the ETL and the SPM methods indicate that the design deepwater significant wave is 3.6 feet high, which comes from the East fetch (1.10 miles long) and is produced by a 72 miles per hour (mph) wind of 15 minute duration. It has a wave length of 40 feet and a period of 2.8 seconds.

j. A simple triangular approximation of the profile of the east fetch indicates that 96 percent of the fetch length has water depths at maximum reservoir water surface elevation deeper than 20 feet. It was therefore concluded that the predominant depth of water was deeper than half of the design deepwater significant wave length and that shallow water wave heights would not have to be considered.

k. Paragraph 2-13 of EM 1110-2-1614 indicates that runup on earth embankments armored with riprap can be estimated with equation 3 :

$R/H_s = 1/(0.4+(H_s/L_o)^{0.5} \cot(\text{theta}))$

where:

R is the wave runup above the still water line

H_s is design significant wave height (3.6 feet)

L_o is the design wave length (40.1 feet), and

cot(theta) is the horizontal component of a sideslope with a unit vertical component.

This equation produces a wave runup of 3.6 feet.

WIND SETUP

5-62 Reference 4, indicates that equation 3-97 in the 1977 edition of the SPM should be used to determine the wind setup:

$\text{delta S} = (CW^2F \cos(\text{theta}))/D$

where:

delta S is the wind setup
D is the average depth of the fetch (252 feet)
F is the fetch length (1.10 miles)
W is the wind velocity (72.3 mph)
theta is the angle between the wind and fetch (0), and
C is 1.165

This equation produces a wind setup of 0.03 feet.

RESULTS

5-63 The resulting top of dam elevations for the various cases are as follows:

a. FRL B + 3.0 feet is 2,607.4 feet NGVD.

b. FRL B + Wave Runup + Wind Setup is 2,608.0 feet NGVD.

c. FRL C + 5.0 feet is 2,609.4 feet NGVD.

Case c governs. This value was rounded up to 2,610 feet NGVD.

Minimum Revetment Design

GENERAL

5-64 The hydraulic design of the minimum protection for the upstream side of the dam embankment is generally based upon EM 1110-2-2300 (ref. 9). The guidance given in this EM, however, was somewhat modified in accordance with instructions contained in reference 4.

EMBANKMENT ZONES

5-65 As indicated in Appendix C of EM 1110-2-2300, the upstream slope of the proposed Seven Oaks Dam embankment was divided into four zones.

5-66 The class I zone extends from an upper pool elevation with a 10 percent chance of exceedence plus wave runup, to a lower pool elevation determined by the drawdown pool with a 10 percent chance of occurrence. At Seven Oaks Dam the pool elevation with a 10 percent chance of exceedence is elevation 2,367 feet NGVD. Wave runup (see paragraph 5-61 (k) above for the method of computation) is 4.9 feet. Rounding up, this produces a top elevation for the class I zone of 2,372 feet NGVD. Since Seven Oaks dam will be drained dry every year, the lower limit of the class I zone is the embankment toe at elevation 2,100 feet NGVD.

5-67 The class II zone extends from the uncontrolled spillway crest plus wave runup down to the class I zone. In the case of Seven Oaks Dam the spillway crest is at elevation 2,580 feet NGVD, and the wave runup (see paragraph 5-61 (k) above for method of computation) is 3.9 feet. Rounding up, this gives a top elevation of 2,584 feet NGVD for the class II zone.

5-68 The class III zone extends from the special zone near the top of the dam to the top of the class II zone.

5-69 The special zone is the zone at the top of the dam embankment that protects the dam crest from a spillway design flood. This zone was assumed to extend from the top of the dam to the maximum reservoir water surface during the spillway design flood minus the coincident wave height. The top of dam is at elevation 2,610 feet NGVD. The maximum reservoir water surface during the spillway design flood is elevation 2,604.4 feet NGVD. The significant wave height coincident with the maximum reservoir water surface was determined to be 3.6 feet as explained in paragraph 5-61 (i) above. The special zone was therefore established to extend from 2,610 to 2,600 feet NGVD (the lower elevation was rounded down from 2,600.5 feet NGVD).

DESIGN PARAMETERS

Design Wind

5-70 EM 1110-2-2300 indicates that the design wind for class I design should be the 1 percent chance, or 100-year, wind. A wind rose of the 100-year maximum one-hour overland winds at the Seven Oaks Dam site was used to obtain the 100-year maximum one-hour overland wind at the Seven

Oaks Dam site. The wind rose was developed from hydrometeorological analysis of historical wind data at nearby Norton Air Force Base, with allowance made for differences in site conditions at Seven Oaks Dam.

5-71 EM 1110-2-2300 indicates that the design wind for class II design should be the 50 percent chance or the 2-year wind. A wind rose was used to obtain the 2-year maximum one-hour overland wind at the Seven Oaks Dam site.

Effective Fetch

5-72 EM 1110-2-2300 recommends that the procedure outlined in Figure 10 of ETL 1110-2-221 (ref. 10) be used for computing the effective fetch. Reference 4., however, indicates that the procedure outlined in ETL 1110-2-305 (ref. 5) should be used. The method in ETL 1110-2-305 was used, and is described below.

5-73 EM 1110-2-2300 indicates that the fetch to be used for class I design is the effective fetch for a pool with a 10 percent chance of exceedence. Design fetch lengths were determined for the East, East-northeast, Northeast, North-northeast and North directions. Figure V-3 from the Phase I GDM Supplement, Main Report (ref. 16) was used as the basis for the fetch lengths. As indicated in ETL 1110-2-305, the design fetch length was taken as the arithmetic average of nine fetches measured over a 24 degree sector centered around the nominal wind direction. The fetches were measured from the top of the dam embankment to the end of the maximum reservoir pool. The location on the top of the dam embankment from which the fetches were measured was located by visual estimate so that the average fetch length would be maximized.

5-74 EM 1110-2-2300 indicates that the fetch to be used for class II design is the effective fetch for a pool at spillway crest. This was assumed to be approximately equal to the maximum reservoir water surface elevation.

Design Wave

5-75 EM 1110-2-2300 indicates that the procedure outlined on Figure 11 of ETL 1110-2-221 should be used for determining significant wave height. Reference 4, however, indicates that the procedure in ETL 1110-2-305 should be used. The method in ETL 1110-2-305 was used and is described in detail in paragraphs 5-77 to 5-81 below.

5-76 EM 1110-2-2300 indicates that the slope protection provided at the near crest elevations must be stable for the design wave used to establish the top of dam elevation. In the case of Seven Oaks Dam, the design wave height did not govern the top of dam elevation. It was therefore determined that the more conservative of either the coincident wave for the maximum reservoir water surface, or the wave used in the class II design would be used. The wave height for the class II design turned out to be more conservative.

Significant Wave Height

5-77 The fastest mile, maximum two-hour, three-hour, four-hour and six-hour winds for the 100 and 2-year winds were obtained by multiplying the maximum one-hour overland wind obtained from the wind roses by 1.25, 0.96, 0.93, 0.91 and 0.88 respectively.

5-78 To convert to overwater wind speeds, all the overland wind speeds were multiplied by 1.1. None of the fetch lengths were longer than 10 miles, so no attempt was made to compute a wind stability correction factor based upon air and water temperatures.

5-79 For the same reasons discussed in paragraph 5-61 (h) above the Shore Protection Manual (ref. 7) was used instead of ETL 1110-2-305 to determine the deepwater significant wave heights.

5-80 The results indicate that the class I design deepwater significant wave is 5.3 feet high, comes from the east-northeast fetch (0.76 miles long), is produced by a 115 mph wind of 12 minute duration, and has a wave length of 46 feet and a period of 3.0 seconds. The class II design deepwater significant wave is 3.9 feet high, comes from the east-northeast fetch (1.08 miles long), is produced by a 78 mph wind of 15 minute duration, and has a wave length of 42 feet and a period of 2.9 seconds.

5-81 By inspection, it is obvious for both class I and class II designs that most of the fetch length has water depths deeper than 21-23 feet. It was therefore concluded that the predominant depth of water was deeper than half of the design deepwater significant wave length and that shallow water wave heights would not have to be considered.

Wave Runup

5-82 Refer to paragraph 5-61 (k) above for a discussion of the method for computing wave runup. The wave runup was determined to be 4.9 feet for the class I design, and 3.9 feet for the class II design.

LAYER THICKNESS AND STONE SIZE

Design Procedure

5-83 Appendix C of EM 1110-2-2300 indicates that the following equations should be used for determining layer thickness and stone size.

$$W_a = \frac{\text{gamma}H_s^3}{4.37\ \cot(\text{alpha})(G-1)^3}$$

$$W_{max} = 4W_a$$

$$W_{min} = \frac{W_a}{8}$$

$T = 20\ (W_a/gamma)^{1/3}$

where:

W_a = weight of the median sized stone (lbs)
W_{max} = weight of the maximum sized stone (lbs)
W_{min} = weight of the minimum sized stone (lbs)
T = thickness of the riprap layer (inches)
gamma = unit weight of the stone (lbs/ft3)
H_s = significant wave height (ft)
alpha = slope angle from the horizontal (degrees)
G = specific gravity of the stone material

5-84 Assuming spherical stone, the following equation from paragraph 14d of EM 1110-2-1601 (ref. 17.) was used to estimate stone diameters:

$$D = (6W/pi*gamma)^{1/3}$$

where D = stone diameter

Results

5-85 The results indicate that the class I zone will require a 31-inch layer of riprap with maximum, median and minimum stone diameters of 36, 23 and 12 inches, respectively. Since the layer thickness cannot be less than the maximum rock size (guidance in EM 1110-2-1601), the minimum required layer thickness is 36 inches.

5-86 The results indicate that the class II zone will require a 23-inch layer of riprap with maximum, median and minimum stone diameters of 27, 17 and 9 inches, respectively. For the reasons discussed above, the minimum required layer thickness is 27 inches.

5-87 According to EM 1110-2-2300, the Class III zone requires no riprap.

5-88 As indicated in paragraph 5-76 above, the class II design wave was used for the special zone riprap design. Since the class II design was performed using the fetches for the maximum reservoir water surface, all the design parameters for the special zone riprap design are the same as those for the class II design. The riprap for the special zone, therefore, has the same layer thickness and stone size as in the class II zone.

5-89 The results discussed above pertain only to the hydraulic design of the minimum revetment required to protect the upstream side of the dam embankment. However, because of geotechnical considerations, the actual rock layer covering the upstream side of the embankment will be considerably thicker than the minimum thickness based on hydraulic criteria. Further discussion is provided in Appendix A, Geotechnical.

Diversion of Water During Construction for Upstream Control Alternative

5-90 The hydraulic design of the diversion of water during construction for the upstream control alternative was based on the same criteria and procedures used for the downstream control alternative, as discussed in detail in paragraphs 5-94 through 5-101 below. A complete description of the diversion plan is provided in Section XIII, Diversion and Control of Water During Construction. Basically, all diversion of flows will take place through the outlet works tunnel, which will be an 18-foot diameter, inverted horseshoe. An elevation-discharge rating curve for this tunnel was developed considering both inlet and outlet control. The curve was developed assuming a 26-foot diameter, semi-circular conduit, which was the configuration under consideration at the time the analysis was prepared. The two configurations are hydraulically equivalent. The rating curve is displayed in figure 11. Using this curve, various frequency floods were routed through the reservoir to determine the corresponding maximum pool elevation. The results of the routings were used to prepare the curve of level of protection versus cofferdam height shown in figure 12. From this curve, a 50-foot high cofferdam will provide an 18-year level of protection.

Diversion of Water During Construction for Downstream Control Alternative

GENERAL

5-91 Four different methods of diversion will be employed at the Seven Oaks Dam construction site in five phases. Initially, a 50-foot-high cofferdam spanning the upper Santa Ana canyon in combination with a concrete lined diversion tunnel will be used. This will protect the main embankment foundation excavation and the early stages of the main embankment fill.

5-92 Eventually the main embankment fill will rise above and incorporate the original 50-foot-high cofferdam. When the embankment has reached the desired height above streambed, a steel penstock will be laid within the original concrete lined diversion tunnel. This construction will take place during the dry season when flood events are much less frequent. At this time the diversion will be carried out by bulkheading off the concrete diversion tunnel and diverting into two steel conduits. These steel conduits will be buried below the invert of the concrete diversion tunnel when it is originally constructed and eventually will become the low flow by-pass and the minimum discharge line. Also refer to outlet works, downstream control alternative, discussion in Appendix B.

5-93 After a single dry season, the steel penstock will be complete. The bulkhead at the inlet to the diversion tunnel (now the inlet to the steel penstock) will be removed and the wet season diversion will take place through the steel penstock.

5-94 When the next dry season arrives, a bulkhead will be placed at the inlet to the steel penstock and diversion again will take place through the steel pipes buried below the invert of the former diversion tunnel. At this time the construction of the downstream gate facilities will take place.

5-95 After another single dry season the outlet works will be complete and the bulkhead on the inlet will be removed. Any further diversion that is required will be performed by operating the gates. A complete discussion of the diversion of water during construction is provided in Section XIII.

DIVERSION TUNNEL

5-96 The diversion tunnel will consist of a concrete lined conduit with a total length of approximately 1,680 feet. At the upstream end the tunnel will have a bell-mouthed entrance on the sides and top. The tunnel cross section at the entrance will be horseshoe shaped with a height and width of 18 feet. The concrete lining will have a high velocity finish.

Elevation-Discharge Rating Curve

Inlet Control Analysis

5-97 Critical Depth was assumed to analyze inlet control for depths less than 0.8 times the intake height, (i.e.: (.8)(18) = 14.4 feet). The orifice equation was used to analyze inlet control for depths greater than 1.2 times the intake height, (i.e.: (1.2)(18) = 21.6 feet). An orifice coefficient of 0.95 was assumed. For depths between 0.8 and 1.2 times the intake height, a curve was interpolated between the above curves.

Outlet Control Analysis

5-98 The intake loss was assumed to be 0.06 times the velocity head in the tunnel just downstream from the entrance. Friction losses in the conduit were estimated using the Darcy-Weisbach equation. An absolute roughness of 0.002 feet was selected based upon paragraph D-3 of EM 1110-2-1602. The Colebrook-White equation was used to obtain the Darcy-Weisbach friction factor.

Results

5-99 The resulting rating curves indicated that inlet control governs up to a reservoir water surface elevation of approximately 2,160 feet NGVD. For elevations above this, outlet control governs. See figure 4 for the Elevation versus Discharge rating curve.

Elevation-Level of Protection Rating Curve

Reservoir Routings

5-100 Reservoir routings were performed and maximum reservoir water surfaces were computed for the 2, 5, 10, 15, 20, 25, 50 and 100 year flood hydrographs. The results were used to plot curves of level of protection versus cofferdam height.

Results

5-101 The results indicate that for a 50-foot-high cofferdam there will be an 18-year level of protection to the embankment foundation excavation. See figure 5 for the entire Elevation versus Level of Protection Rating Curve.

STEEL PIPES

5-102 There will be two steel pipes laid beneath the invert of the concrete lined diversion tunnel. One will have an inside diameter of 3'-3" and the other will have an inside diameter of 3'-6". For purposes of estimating the capacity of the two pipes, they were assumed to be identical except for their diameters. Each was assumed to be a continuous steel pipe with no coating and with an approximate length of 1,730 feet. At the upstream end a full bell-mouthed entrance was assumed for each. Each pipe was assumed to have one 48 degree bend near the upstream end and one 14 degree bend near the downstream end. At the downstream end each pipe was assumed to transition to a rectangular passage (2' x 3') with two sets of gate slots.

Elevation-Discharge Rating Curve

Energy Losses

5-103 The intake loss for the 3'-3" and 3'-6" pipes was assumed to be 0.12 times the velocity head in the tunnel just downstream from the entrance. King's Handbook (ref. 18) recommends values varying from 0.04 to 0.20. The average was selected. Bend losses were estimated using Plate C-10 of EM 1110-2-1602. The transition loss was estimated as follows. King's Handbook indicates that for a sudden contraction between the 3'-3" pipe and the 2' x 3' passage, the loss varies from 0.07 to 0.11 times the velocity head in the upstream conduit. The loss was assumed to be half of the greater value: .11/2 = 0.06. The reasoning is that the loss would be somewhere between sudden and negligible and that using the value at the upper end of the range of losses would be conservative. The same loss was assumed in the 3'-6" pipe. The losses through the gate slots were assumed to be 0.01 times the velocity head through each of the two slots. This is based upon paragraph 3-13 of EM 1110-2-1602. Friction losses in the conduit were estimated using the Darcy-Weisbach equation. An absolute roughness of 0.0006 feet was selected based upon paragraph 2-12 g.(2)(a) of EM 1110-2-1602. The Colebrook-White equation was used to obtain the Darcy-Weisbach friction factor.

Results

5-104 See figures 6 and 7 for the Elevation versus Discharge rating curves.

Elevation-Level of Protection Rating Curve

Reservoir Routings

5-105 Reservoir routings were performed and maximum reservoir water surfaces were computed for the 2-, 5-, 10-, 20-, and 100-year summer flood hydrographs. The results were used to plot a curve of level of protection versus cofferdam height.

Results

5-106 See figure 8 for the resulting Elevation versus Level of Protection Curve.

STEEL PENSTOCK

5-107 The steel penstock consists of a continuous unlined steel pipe with an inside diameter of 11 feet. The length was assumed to be approximately 1,580 feet. At the upstream end, the inlet has a full bell-mouthed entrance.

Elevation-Discharge Rating Curve

Inlet Control Analysis

5-108 Critical Depth was assumed for analysis of inlet control for depths less than 0.8 times the intake height, i.e.: (.8)(11) = 8.8 feet. The orifice equation was used to analyze inlet control for depths greater than 1.2 times the intake height, i.e.: (1.2)(11) = 13.2 feet. An orifice coefficient of 0.95 was assumed. For depths between 0.8 and 1.2 times the intake height, a curve was interpolated between the above curves.

Outlet Control Analysis

5-109 The intake loss was assumed to be 0.06 times the velocity head in the tunnel just downstream from the entrance. Friction losses in the conduit were estimated using the Darcy-Weisbach equation. An absolute roughness of 0.0006 feet was selected based upon paragraph 2-12.g.(2)(a) of EM 1110-2-1602. The Colebrook-White equation was used to obtain the Darcy-Weisbach friction factor.

Results

5-110 The resulting rating curves indicated that inlet control governs up to a reservoir water surface elevation of approximately 2,128 feet NGVD. For elevations above this, outlet control governs. See figure 9 for the Elevation versus Discharge rating curve.

Elevation-Level of Protection Rating Curve

Reservoir Routings

5-111 Reservoir routings were performed and maximum reservoir water surfaces were computed for the 2, 5, 10, 15, 20, 25, 50 and 100 year flood hydrographs. The results were used to plot curves of level of protection versus cofferdam height.

Results

5-112 See figure 10 for the entire Elevation versus Level of Protection Rating Curve.

Diversion of Water During Construction for Mid-Tunnel Control Alternative

5-113 The regulating outlet tunnel for the mid-tunnel control alternative would be used for diversion of water during construction and would have the same configuration as for the downstream control alternative before placement of the steel penstock. For both alternatives, the tunnel would consist of a 18-foot diameter horseshoe section. There would be some difference in the transitions upstream and downstream of the gates, but the energy loss through the transitions for downstream control was found to be negligible. As a result, the curve of level of protection versus cofferdam height for downstream control (fig. 5) is also applicable for mid-tunnel control. A complete discussion of the diversion plan for the mid-tunnel control alternative is presented in Section XIII.

MAIN COFFERDAM

5-114 The optimum main cofferdam height was determined to be 50 feet, based upon minimizing the risk of flooding to downstream interests during the initial phases of dam construction. See Appendix E, Seven Oaks Dam Inundation Study, for a complete description of the determination of the main cofferdam height.

GOVERNMENT CANYON

5-115 The confluence of Government Canyon with the Santa Ana River occurs within the foundation excavation area downstream from the main cofferdam. Flows from Government Canyon must therefore also be diverted from inundating the foundation excavation. It was decided that a 25-year level of protection would be the minimum desireable and that the cofferdam should not exceed 50 feet in height.

Alternatives

Cofferdam and Pipeline

5-116 General. Two alternative cofferdam and outlet pipe alternatives were analyzed. Both alternatives featured a cofferdam on Government Canyon, and a 2,310-foot-long corrugated metal outlet pipe draining through the cofferdam embankment to a location just downstream from the proposed main embankment foundation excavation. In one alternative the pipe was 3 feet in diameter, and in the other it was 5 feet in diameter. Both pipes had approximately seventeen bends. Deflection angles in the bends varied from 2 to 141 degrees. A projecting entrance was assumed for the pipes. Outlet control was assumed for both pipes.

5-117 Energy Losses. An entrance loss coefficient of 0.90 times the velocity head in the pipe was assumed based on a value recommended in King's Handbook (ref. 18) for projecting entrances. Friction losses were determined using plate C-7 in EM 1110-2-1602 (ref. 19). A 90 degree helix was assumed for the corrugated metal pipes. This method resulted in a Darcy-Weisbach f value of 0.064 for the 5-foot diameter pipe, and a value of 0.079 for the 3-foot diameter pipe. Bend losses were determined using plate C-10 in EM 1110-2-1602. It was assumed that all bends would be mitered. This resulted in a total loss coefficient for all bends of 4.52 times the velocity head in the pipe.

5-118 Rating Curves. Using the above determined loss coefficients, rating curves were developed for both pipes. When a 50-foot-high cofferdam would be full of water, the 3-foot pipe would have a discharge capacity of 76 ft^3/s, and the 5-foot pipe would have a discharge capacity of 287 ft^3/s.

5-119 Routings. Upon routing the 25-year flood it was determined that with a 50-foot-high cofferdam, neither of the two pipeline alternatives could provide a 25-year level of protection (the 25-year flood from Government Canyon has a peak discharge of 700 ft^3/s).

Cofferdam and Drainage Channel

5-120 General. This alternative consists of a cofferdam on Government Canyon upstream from the main embankment foundation excavation, and a drainage channel excavated between Government Canyon and the Santa Ana River just upstream of the Government Canyon cofferdam. This would connect the two drainage areas, and flows from Government Canyon would also drain through the outlet works diversion tunnel in the left abutment (pl. 23).

5-121 Analysis. As a first trial, it was assumed that the minimum constructable drainage channel would have a base width of 10 feet. The proposed channel is assumed to have cut slopes of 2V on 1H, and is approximately 700 feet long. The channel is cut from the invert of Government Canyon to the invert of the Santa Ana River, which results in a slope of 0.0534. By checking normal and critical depth in both the drainage channel and Government Canyon upstream from the cofferdam it was determined that both would flow in the super-critical regime (a Manning's roughness coefficient of 0.035 was used for both). It was assumed that a pool would form behind the cofferdam and that flows coming down Government Canyon would pass through a hydraulic jump when entering the pool. It was further assumed that flows draining out of the pool through the drainage channel would pass through critical depth at some point in the channel near the upstream end (i.e. so that friction losses between this point and the pool would be minimal). The pool elevation for a discharge of 700 ft^3/s was determined by computing critical depth in the drainage channel, and adding the velocity head to it. It was further assumed that conditions in the pool would be rather turbulent. The height of waves forming in the pool was assumed to be equal to the height of the hydraulic jump which would occur when flows from Government Canyon enter the pool. The wave runup on the cofferdam embankment was assumed to be the same as the wave height. The resulting necessary cofferdam height was 22.29 feet. A 50-foot-high cofferdam in conjunction with the proposed drainage channel would therefore provide better than the 25-year level of protection. On the possibility that a pool would not form behind the cofferdam, an additional check was performed. In this case flows from Government Canyon would impact directly against the cofferdam. The maximum runup on the cofferdam such flows would reach was estimated by computing the energy grade line assuming normal depth for 700 ft^3/s. It was determined that for this case the cofferdam would have to be 13.63 feet high.

Selected Design

5-122 The selected plan for protecting the main embankment foundation excavation is a cofferdam at least 23 feet high in Government Canyon in conjunction with a drainage channel through the Government Canyon Ridge. As discussed in Section XIII, the cofferdam height for Government Canyon was set at 50 feet. Also, for ease of construction, the base width of the proposed drainage channel was increased from 10 feet to 30 feet, and the side slopes were flattened to 1V on 1H with benches. The invert slope will remain at approximately 0.05. The revised channel design will provide a greater capacity than the capacity based on the original design, which was more than adequate. Consequently, no further hydraulic analysis or design was required for the larger drainage channel. See plate 23 for the locations of these features.

FLOODWAY ESTIMATES

General

5-123 The delineation of a 100-year post-project floodway on the Santa Ana River between Prado Dam and the Seven Oaks damsite is a project feature of the Santa Ana River project associated with the proposed Seven Oaks Dam. The delineation of pre- and post-project floodways is also required for environmental analysis of the proposed project. For the Phase II General Design Memorandum, however, it was not possible to accurately delineate the floodways due to a lack of adequate topographic mapping. Instead, estimates of the floodways were made using the best available mapping; the estimates are adequate for the purposes of environmental evaluation and project cost estimation. A final detailed post-project floodway and 100-year floodplain will be delineated and documented in a Feature Design Memorandum.

5-124 The Santa Ana River between Prado Dam and the proposed Seven Oaks damsite was divided into three distinct reaches, which were analyzed separately. The first reach extends from Prado Dam to the downstream end of the Lytle-Warm Creek confluence with the Santa Ana River. The second reach is the Lytle-Warm Creek confluence, and the third reach extends from the upstream end of the Lytle-Warm Creek confluence to the Seven Oaks damsite.

Prado Dam to Lytle-Warm Creek Confluence

Reach Description

5-125 This reach extends from River Road (at the upstream end of the Prado Dam Flood Control Basin) to 1,000 feet downstream of the Mt. Vernon Avenue Bridge (the downstream end of the Corps' Lytle-Warm Creek Project). This reach is about 21 miles long and the channel bottom throughout the reach is generally sandy with some finer and coarser material present. The channel slopes range from about 0.003 to 0.006 with an average of 0.004. Levees exist on both banks from downstream from Mission Blvd. to Crestmore Road. There is a single levee on the south bank from Crestmore Road to Riverside Avenue. The channel width in the study reach varies from 20 to 1,000 feet. Through Riverside Narrows, the low flow channel winds back and forth between canyon slopes with the overall river channel usually staying 150-feet in width or less.

Hydrology

5-126 The following 100-year present condition discharges (ft^3/s) were used:

Location	w/o project	w/ project
At E Street	105,000	67,000
At Riverside Narrows	175,000	130,000

Method of Analysis

5-127 The HEC-2 "Water Surface Profiles" computer program was used to compute the with- and without-project floodways for this reach. Both analyses included the development of (1) the natural, 100-year flood event water surface profile, and (2) an "encroached" profile. The flood boundaries of the natural profile were encroached until a one foot rise in the water surface was produced to create the second or "encroached" profile. In the cases where less than a one foot rise was achieved by the encroachments, the encroachment stations were moved right up to, but not allowed within, the channel bank stations. The bank stations were assumed to be the low flow channel banks of the river as indicated by the stippled area on USGS topographic quadrangle sheets. During initial trial encroachments, at some cross sections the rise in the encroached water surface elevation tended to be greater than one foot due to the backwater effect of downstream encroachments. This sometimes occurred even when there was no encroachment applied to that particular cross section. It was determined that the maximum increase in water surface elevation due to encroachment should be 1.5 feet. In order to limit the increase in water surface to this amount, downstream encroachments sometimes had to be "eased off." As a result of this, in several cross-section locations, encroachment on the natural floodway yielded much less than one foot of rise in the water surface elevation.

Cross Sections

5-128 The geometry of the cross sections for this reach required by the HEC-2 computer model was determined from USGS topographic quadrangle sheets. The distance between cross sections normally varied between 1,000 and 3,000 feet. The channel bank stations were taken to be the low flow channel boundaries on the USGS maps. The low flow river channels on these maps are indicated by a stippled area.

Bridges

5-129 Twelve bridges were located in this study reach. One bridge, River Road Bridge, is located in Prado Basin and was assumed to wash out or have a negligible impact upon the 100-year flow, so it was not modeled. This assumption is based on water surface elevation information found in the flood insurance reports for Riverside County. Previously existing bridge models for seven bridges: Hamner Avenue, Crestmore Road, Riverside Avenue, 8th Street, the Atchison Topeka, and Santa Fe (AT & SF) Railroad, the Southern Pacific (SP) Railroad, and Mt. Vernon Avenue were taken from HEC-2 models developed outside the Corps. These models were substituted into the HEC-2 model without changes or any checks for accuracy, since no plans for these bridges were available. Three of the four remaining bridges, Mission Boulevard, Highway 60, and the Union Pacific (UP) Railroad, were modeled from available plans. The remaining bridge, Van Buren Boulevard, is a double bridge. The north bound side was modeled from available plans and the south bound side was modeled from field measurements.

Roughness Coefficients

5-130 The Manning's Roughness Coefficients ("n" values) for this study reach were assumed to be 0.035. This value was selected based upon engineering judgement, since analytical methods did not produce consistent results. Refinement of the "n" values will be made for the final floodway, including investigation of alternative analytical methods.

Downstream Water Surface Elevations

5-131 The downstream water surface elevation for the natural profiles was estimated by the HEC-2 computer program using the slope-area method. The average slope of the channel reach was determined from the USGS topographic quadrangle sheet to be 0.004. The starting water surface elevation for the encroached profiles was set one foot higher than that for the natural profiles.

Lytle-Warm Creek Confluence

5-132 The Lytle-Warm Creek confluence encompasses an approximately 1.7 mile long reach of the Santa Ana River. Within this reach the Los Angeles District Corps of Engineers has built substantial channel improvements to confine flows and protect ten bridges. For purposes of estimating floodways, it was simply assumed that the boundaries would coincide with those of the Corps' channel and no further analysis was performed.

Lytle-Warm Creek Confluence to Seven Oaks Damsite

Reach Description

5-133 The reach covered in the upper part of the analysis extends from the upstream end of the Corps of Engineers' Lytle- Warm Creek project to the proposed Seven Oaks damsite. This reach is about 12 miles long. Throughout the reach, channel slopes are fairly steep and bed material is generally coarse. At the downstream end of this reach, the channel consists of a soft bottom channel with levees on both sides. The channel is about 1,300 feet wide with an invert slope of about 0.007. Approximately 4.3 miles upstream from the downstream end, the river becomes part of a broad wash. This wash is up to 8,000 feet wide and is bounded by high ground on the north side and by well defined high banks on the south side. At present, the low flow channel of the river runs along the southern side of the wash, and the low flow channels of Plunge and City Creeks run along the north side of the wash. The invert slope of the Santa Ana River low flow channel varies from about .016 at the downstream end of the wash to .022 at the upstream end of the wash. The wash narrows down and enters the Upper Santa Ana canyon about 6.7 miles upstream from the downstream end of the wash. In the Upper Santa Ana Canyon the river is natural. It is about 600 feet wide and is bounded by steep canyon walls. The river in this area has an invert slope of about 0.028.

Method of Analysis

5-134 The with- and without-project floodways were computed using the HEC-2 "Water Surface Profiles" computer program as described in paragraph 5-124. At some cross sections, the encroachments did not produce a one foot rise in water surface even when they encroached all the way to the channel banks. In such cases the encroachments were not allowed to intrude further than the bank stations. At other cross sections, greater than one foot water surface elevations resulted. This occurred even though the encroachments were set at the natural overflow boundary. This was due to the backwater effect of downstream encroachments. Although it would be possible to reduce the rise to one foot by moving the encroachments outward, this was not done as it would produce a floodway which would be wider than the natural flood boundaries. If a greater level of detail had been required, more effort would normally have been expended to resolve such anomalies; however, the approach taken is deemed adequate for the given purpose.

Hydrology

5-135 The following 100-year present condition discharges (ft^3/s) were used:

Location	w/o project	w/project
At Seven Oaks	58,000	5,000
At Mill Creek	75,000	25,000
At City Creek	80,000	32,000

Cross Sections

5-136 The cross sections required by the HEC-2 computer model were obtained from USGS Topographic Quadrangle sheets. They were taken approximately 1,000 to 3,000 feet apart. Channel bank stations were assumed to coincide with the boundaries of the stippled areas which the USGS uses to indicate river low flow channels. It was assumed that the levees protecting Norton Air Force base would not overtop and that flows would generally remain along the south side of the wash.

Bridges

5-137 Nine bridges have been located in this part of the study reach. Several bridges were modeled by simply substituting pre-existing HEC-2 bridge models. The Waterman Avenue, Santa Fe Railroad and Tippecanoe Avenue models were taken from HEC-2 models produced outside the Corps. As no bridge plans were available, no attempt was made to check the models for accuracy. The model of the abandoned railroad bridge near Norton Air Force Base was adapted from a Flood Hazard Study and was originally based upon field measurements. The Highway 30, Orange Street and Greenspot Road bridges were modelled based upon field measurements. It was assumed that the Alabama Street crossing in the wash and the Southern California Edison Co. bridge and tailrace in the canyon would have a negligible impact upon flows and were not modelled.

Roughness Coefficients

5-138 Manning's Roughness Coefficients ("n" values) were selected using equation (9) from reference 14. Based upon this equation "n" values were computed which varied from .040 at the downstream end of the study reach to 0.070 at the upstream end.

Downstream Water Surface Elevations

5-139 For the natural profiles, the downstream water surface elevation was estimated by the HEC-2 program using the slope-area method. For the encroached profiles the starting water surface elevation was set one foot higher than the natural profile starting water surface elevation.

Results

5-140 The resulting with-project floodway is shown on plates 34 and 35. Given the level of detail and the many compromising assumptions that were used to develop this information, it is strongly recommended that its use be strictly limited to the environmental analysis.

OVERFLOW BOUNDARY ESTIMATES

General

5-141 For the purpose of estimating the environmental impact of Seven Oaks Dam on plants growing in the Santa Ana Wash, the estimated with- and without-project 5, 10, 25, 50 and 100-year flood boundaries were delineated between the proposed Seven Oaks Dams and Norton Air Force Base (AFB). Refer to paragraph 5-133 above, for a description of the study reach.

Method of Analysis

5-142 The with- and without-project flood boundaries were computed using the HEC-2 "Water Surface Profiles" computer program. For both the with- and without-project analyses, the natural, present condition HEC-2 model developed for previous floodway estimates was adapted by substituting future condition hydrology values into the model. This produced two new models, one representing the with-project conditions and one representing the without-project conditions. No other modifications were attempted. Refer to paragraphs 5-135 through 5-139 above for a detailed description of this model.

Hydrology

5-143 The following without-project future condition peak discharges (in ft^3/s) on the Santa Ana River were used:

Location	5-yr	10-yr	25-yr	50-yr	100-yr
Seven Oaks	4300	8800	20500	34000	58000
Mill Creek	5600	11700	26000	45000	76000
City Creek	5800	12500	28000	48000	80000

The following with-project future condition peak discharges (ft^3/s) on the Santa Ana River were used:

Location	5-yr	10-yr	25-yr	50-yr	100-yr
Seven Oaks	2000	2150	3150	4200	5500
Mill Creek	2050	4300	9300	15500	25000
City Creek	2600	5600	13000	21000	33000

Subsequent to the time the overflow analysis was prepared, some of the above discharges were revised slightly. The revised discharges are shown in Volume 7, Hydrology, of the Phase II GDM. The revisions to the discharges are too small to significantly affect the location of the flood boundaries, especially in light of the limitations of the available topographic mapping. Consequently, the overflow maps were not revised.

Results

5-144 The resulting with- and without-project flood boundaries are on file in the Hydraulics Section of the Los Angeles District Corps of Engineers.

SEDIMENT TRANSPORT

5-145 Refer to Appendix G for a detailed description of the sediment transport analysis performed for the reach from Prado Dam to the proposed Seven Oaks damsite.

References

1. EC 1110-2-241, EM 1110-2-4000 Appendix E "Sediment Distribution in Reservoirs", 5 November 1982.

2. EM 1110-2-1603, Hydraulic Design of Spillways, 31 March 1965.

3. "Tests of Broad Crested Weirs, Appendix: Supplemental Tests-Weirs with Aprons Inclined Up Stream and Down Stream", Alex R. Webb, ASCE Transactions, Volume 96, 1932.

4. 2nd Endorsement to Letter (10 August 1984) from DAEN-CWH--Y/D (HUFFMAN/MUNSEY), SUBJECT: Redbank and Fancher Creeks Project, Freeboard and Riprap Considerations, to Commander, South Pacific Division, ATTN: SPDED-W, dated 21 November 1984.

5. ETL 1110-2-305, "Determining Sheltered Water Wave Characteristics", Department of the Army, U.S. Army Corps of Engineers, Office of the Chief of Engineers, September 1983.

6. EM 1110-2-1614, "Design of Coastal Revetments, Seawalls, and Bulkheads", Department of the Army, Corps of Engineers, Office of the Chief of Engineers, 30 April 1985.
Department of the Army, Waterways Experiment Station, Vicksburg, Mississippi, 1984.

8. "Shore Protection Manual", Coastal Engineering Research Center, Department of the Army, 1977.

9. EM 1110-2-2300, "Engineering and Design, Earth and Rock Fill Dams, General Design and Construction Considerations", Department of the Army, Corps of Engineers, Office of the Chief of Engineers, 10 May 1982.

10. ETL 1110-2-221, "Wave Runup and Wind Setup on Reservoir Embankments", Department of the Army, Office of the Chief of Engineers, Washington, DC, November 1976.

11. "Sediment Transport Technology", by Simons and Senturk, published by Water Resources Publications, Fort Collins Colorado, in 1976 (Ref. 2).

12. "Sediment Transport, Volume 12 of Hydrologic Engineering Methods for Water Resources Development", by the Hydrologic Engineering Center, U.S. Army Corps of Engineers, Davis, California, June 1977.

13. EC 1110-2-27 "Policies and Procedures Pertaining to Determination of Spillway Capacities and Freeboard Allowances of Dams", Department of the Army, Office of the Chief of Engineers, August 1966.

14. "Hydraulics of High Gradient Streams", by R.D. Jarrett., published in the ASCE Journal of Hydraulic Engineering, November 1984.

15. "Factors Determining Bed Forms of Alluvial Streams", V. Vanoni, March 1974, ASCE Journal of the Hydraulics Division.

16. "Upper Santa Ana River, Flood Storage Alternatives Study, Supplement to Phase I GDM on the Santa Ana River Main Stem including Santiago Creek", U.S. Army Corps of Engineers, Los Angeles District, December 1985.

17. EM 1110-2-1601, "Hydraulic Design of Flood Control Channels", Department of the Army, Corps of Engineers, Office of the Chief of Engineers, July 1970.

18. "Handbook of Hydraulics," Brater and King, 6th Ed., McGraw-Hill Book Co., 1976.

19. EM 1110-2-1602, "Hydraulic Design of Reservoir Outlet Works", Department of the Army, Corps of Engineers, Office of the Chief of Engineers, October 1980.

20. Course notes from the class "Hydraulic Design of Spillways and Outlet Works", held from March 10, to March 14, 1986 at the Waterways Experiment Station, in Vicksburg, Mississippi.

21. "Hydraulic Design of Stilling Basins and Energy Dissipators, Engineering Monograph Number 25", U.S. Department of the Interior, Bureau of Reclammation, May 1984.

22. "Erosion of Unconsolidated Gravel Beds", S. P. Chee and E. M. Yuen, Canadian Journal of Civil Engineering, September 1985.

VI. GEOLOGY, SOILS AND MATERIALS

General

6-01 The geologic setting and the results of geotechnical investigations of the damsite and construction materials are summarized in the following paragraphs along with a description of the design considerations, methods of analysis, and preliminary construction requirements for the proposed dam and appurtenances. A complete description of the geotechnical studies conducted for the Phase II GDM is presented in Appendix A, Geotechnical, to this volume.

Geologic Setting

6-02 The Seven Oaks damsite is located in upper Santa Ana River canyon which is within the southern frontal margin of the San Bernardino Mountains. These mountains are in the eastern part of the Transverse Ranges physiographic province in southern California. This province is so named because the Transverse Ranges lie across the grain of adjacent physiographic provinces, which are strongly influenced by the northwest-southeast-trending San Andreas fault system. The San Andreas fault system is the master transform along which the Pacific plate, on the west side, moves northwesterly relative to the North American plate, on the east side. The cumulative right-lateral offset movement along this system over the past few million years is estimated to be from 100 to 180 miles.

6-03 In the vicinity of the project, the San Andreas fault splits into two branches. The Seven Oaks damsite is located in the Wilson Creek block, a wedge of basement rock bounded by the North and South Branches. The damsite is approximately one mile upstream of the South Branch, which in the recent past has been the more active branch, and 1/2 mile downstream from the North Branch.

6-04 Rock units exposed at the damsite consist primarily of Precambrian quartzo-feldspathic gneiss that has been intruded by Cretaceous quartz diorite. The gneissic rocks are typically layered and strongly

foliated, and are often highly sheared. The intruded quartz diorite rocks are moderately to strongly jointed and fractured, and slightly foliated. In the San Bernardino Mountains, these rock units occur only within the Wilson Creek block. Quartz monzonite typical of the San Bernardino Mountains occurs north of the North Branch in the upper portion of the reservoir area.

6-05 Overlying the basement rock units in upper Santa Ana River canyon are several colluvial and alluvial units. Dissected remnants of at least three generations of Quaternary-age stream terrace deposits as much as 500,000 years in age can be identified on the canyon walls. The streambed alluvium in the canyon consists of unconsolidated sand, gravel, cobbles, and granitic boulders up to 5 feet and greater in diameter.

Seismicity

6-06 The region surrounding the damsite is seismically active due to the tectonic interaction of the North American plate and the Pacific plate. Relative movement along the boundary between the plates has formed the San Andreas fault, with the action of the movement being primarily right-lateral strike-slip. The relative movement between the crustal plates as well as the movements within the plates causes release of energy in the form of earthquakes, the magnitude and recurrence interval of which are dependant on various factors relating to fault length and type of movement. A maximum credible event of Richter magnitude greater than 8 is postulated for the San Andreas fault. Other major faults in the general southern California region can potentially rupture with maximum credible events of magnitude 6.5 to 7.5.

6-07 A geologic study of prehistoric earthquakes on the San Andreas fault indicates a mean recurrence interval of 150 to 200 years between great earthquakes on segments of this fault. The most recent large event occurred in 1857 and was centered approximately 115 miles from the damsite, in the area known as the "Big Bend." Several earthquake recurrence models indicate that the next major event in southern California will likely be centered on a segment of the fault nearest to the damsite and may be overdue. Peak horizontal accelerations at the damsite from such an event may exceed 0.7g, and the duration of strong shaking in excess of 0.05g is expected to exceed 40 seconds.

6-08 Although no active fault trace has been discovered at the damsite, a major earthquake on the San Andreas fault can be expected to produce sympathetic movement on random shear planes within bedrock under the dam. Assessment of the fault rupture hazard by a recognized technical expert indicates that lateral displacement along the fault plane itself may be on the order of 20 feet, and a conservative design displacement of as much as 4 feet in any direction at the damsite from a magnitude 8+ earthquake on the South Branch of the San Andreas fault should be assumed.

Investigations

6-09 The complete range of geotechnical investigations and design analysis required for the final design of Seven Oaks Dam was not possible within the time and cost constraints of the Phase II GDM. It was agreed, therefore, that Phase II GDM investigations and design activities would attempt to reduce the uncertainty about the site conditions and construction materials that would have a significant impact on the cost of the dam. The remaining required work would be accomplished with construction funds, after approval of the Phase II GDM, and would be reported in the form of Feature Design Memoranda (FDM). To establish design values and prepare a construction cost estimate for the Phase II GDM, the following items were considered to be of primary importance:

a. Depth and configuration of the foundation bedrock surface in the canyon bottom under the proposed embankment and at the outlet works portals.

b. Characteristics of the foundation bedrock.

c. Location and significance of faults and shears.

d. Location of significant landslides.

e. Static and dynamic properties of the streambed alluvium.

f. Sources and properties of construction materials.

g. Excavatability and quality of the bedrock in the spillway and the Government Canyon drainage channel.

6-10 A field and laboratory investigation program to address these items included geologic mapping, literature research, aerial photo interpretation, fault studies, trenching, drilling, geophysical surveys, and detailed in-situ and laboratory testing. A detailed description of the investigations and testing is presented in sections 5 through 7 of Appendix A, Geotechnical, of this volume.

Foundation Conditions

6-11 Drilling and geophysical surveys at the damsite indicate that the Santa Ana River has eroded an asymmetrical channel into the bedrock, the deepest portions of which are approximately 100 feet below the existing streambed ground surface. The bedrock under the dam will be a combination of moderately to highly weathered gneiss and moderately weathered diorite. Both rock types are highly fractured but relatively tight, with rockmass permeabilities generally less than 0.1 ft/day. The rockmass quality does not improve enough with depth to warrant over-excavation of the bedrock foundation surface. It is anticipated that when the foundation bedrock is exposed, a large number of shears will be encountered. However, it is unlikely that major fault zones will be uncovered.

6-12 The geophysical surveys and exploratory drilling indicate that the streambed alluvium which covers the bedrock in the channel appears to be relatively uniform and probably about as dense as modern construction methods could accomplish if it were removed and replaced, except that the materials in the top 20 feet are somewhat less dense. A detailed investigation of the foundation will be conducted in the FDM study phase to verify the suitability of the foundation and the potential for foundation liquefaction during a seismic event.

6-13 A limited investigation of the abutments, outlet works, drainage channel and spillway has shown that the rock types and quality are very similar to those of the streambed foundation bedrock. The gneiss and diorite are highly fractured, moderately to highly weathered with low permeability. The top of the right abutment and portions of the drainage channel and spillway are covered with relatively thin layers of slopewash, topsoil and grassy chapparal vegetation. The lower portion of the right abutment is very steep and the bedrock is exposed. The left abutment, with a shallower slope, has accumulated a relatively deep deposit of colluvium that is over 100 feet thick in the lower portions.

6-14 Although investigations for the outlet works have been limited to the portals, it is anticipated that the outlet works tunnel will be excavated primarily through blocky and seamy diorite. Numerous small shears and possibly 4 or 5 more significant zones are likely to be encountered. The inlet portal will be constructed in a steep diorite rock face of relatively sound and intact rock. The outlet portal site, however, is covered by about 40 feet of colluvium which may conceal more fractured bedrock.

Landslide Potential

6-15 It is likely that operation of the flood control reservoir will cause some mass movements, most of which will consist of the downslope movement of saturated surficial residual soil and loose rock. There is no recognized potential for large scale landsliding which would pose a hazard to the safety of the embankment.

Construction Materials

6-16 Approximately 38,500,000 yds^3 of material is required to construct the embankment. Materials available from required excavation of the foundation, spillway and drainage channel will be alluvial sands, gravels, cobbles, boulders, and fragmented bedrock that will be suitable for use in the shells of the embankment. Impervious core materials and additional free-draining shell materials are available from borrow sources within 3 miles from the dam. Results of tests on impervious and alluvial materials are presented in detail in Appendix A. Tests on rock representative of the spillway and drainage channel excavation will be conducted during the FDM study phase to assess their characteristics as embankment shell material. Concrete aggregate can be processed from streambed alluvium or purchased from commercial sources in the vicinity.

Design Considerations

6-17 The primary considerations in the design of a dam in upper Santa Ana River canyon are the site conditions and foundation materials, proposed operating conditions, and the ability to withstand expected seismic activity. Due to the highly fractured nature of the foundation bedrock and the potential for sympathetic movement during a seismic event, the ability of the dam and outlet works to survive up to 4 feet of movement in the foundation is essential. Construction materials available at the site lend themselves to use as stable embankment materials in this seismic environment. Seven Oaks Dam is designed to be operated as a flood control and debris storage dam, although water conservation is not precluded if operation schedules are changed in the future.

6-18 The seismic criteria for Seven Oaks Dam was developed by:

a. Defining the seismic zones, faults, and source areas

b. Developing and selecting the site or region specific attenuation characteristics

c. Calculating the site ground motions (acceleration, velocity, displacement and duration) and

d. Selecting analogous time-histories and response spectra with appropriate scale factors.

6-19 The maximum credible earthquake is postulated as a magnitude 8+ event on the South Branch of the San Andreas fault. This fault is considered the primary seismic source capable of generating earthquakes that could cause significant ground shaking at the dam and was selected based upon the degree of activity of faulting, magnitude, peak site horizontal bedrock acceleration, duration of strong shaking, and the distance from the dam.

6-20 Given the foregoing seismic environment and the nature of construction materials available at the site, a rolled earth and rockfill embankment design was developed which will produce a conservatively safe dam while utilizing the materials available at the site in the most cost effective manner. The design includes various defensive measures to insure its ability to safely withstand seismic ground motions. The embankment zoning provides resistance to concentrated leaks by placing high strength and erosion resistant materials at the outer shells where they will do the most good. A crack-resistant impervious core and well-graded cohesionless filter and transition zones are designed to be effective in sealing internal cracks which may develop due to differential settlement or earthquake induced movement. The outer shells of free-draining sands, gravels, and rockfill are designed with 1V on 2H outer slopes to provide stability during an earthquake.

6-21 To insure a stable central core and obtain a positive cutoff of groundwater, the streambed alluvium will be excavated to bedrock within a zone defined by a 1V on 1H line from the crest, both upstream and downstream. Elsewhere under the embankment, the top 20 feet of alluvium will be removed before placement of fill materials. The impervious central core will extend from bedrock to within 10 feet of the crest of the dam, and will have a base width approximately one-third of the maximum hydraulic head. Filter zones next to the core will consist of minus 3-inch alluvial material and the upstream shell will consist of minus 12-inch alluvial material. The materials will be processed from the streambed foundation excavation and the downstream pervious borrow area, and the upstream shell will be overlain by a thick layer of oversize stone. The downstream transition zone will consist of unprocessed rock excavated from the spillway and Government Canyon Ridge, with an outer layer of erosion resistant oversize stone from the pervious borrow area.

Analysis

6-22 The stability of Seven Oaks Dam was analyzed only for static loading conditions and potential permanent displacement for this Phase II GDM study. The results of manual calculations using general force equilibrium equations for non-circular sliding surfaces, and computer calculations to locate critical sliding surfaces using Spencer's procedure show that the computed safety factors for all static loading conditions exceed the minimum allowable safety factors by a comfortable margin. Permanent earthquake-induced deformations, estimated with the dynamic displacement analysis procedure originally proposed by Newmark, are expected to be only about 2 feet for the maximum credible design earthquake with no reservoir pool. The seismic stability analyses of the dam was beyond the scope of the Phase II GDM study and will be conducted during the FDM study phase. The embankment seismic stability will be analyzed both statically and dynamically using finite element methods. The static finite element analysis will analyze the static stresses (normal, horizontal, and shear) of the embankment under various reservoir pool conditions (dry, intermediate, and spillway crest). The dynamic finite element analysis will evaluate the embankment response to the design earthquakes under reservoir pool conditions (maximum credible event for dry and intermediate debris storage conditions and probable maximum event for spillway crest conditions). In addition, appropriate permanent embankment displacement analyses will be conducted.

6-23 Preliminary settlement and seepage calculations were conducted to estimate the conditions which might occur during the construction and operation of the dam. Total settlement may be on the order of 20 feet in the core zone, however most of this occurs during construction and post-construction settlement is expected to be only about 5 feet. Seepage quantities and the location of discharge water surfaces were estimated from construction of flow nets for the most critical combinations of reservoir pool conditions. A finite element seepage analysis that calculates the seepage forces in the dam will be conducted in the FDM phase for various reservoir pool conditions.

Construction Requirements

6-24 Diversion and control of groundwater and surface flows will be required for below-streambed excavation, curtain grouting, surface treatment and embankment fill operations. For estimating purposes, it is assumed that groundwater will be controlled with a slurry trench cutoff just upstream of the cofferdam toe. A temporary impervious blanket will be provided on the upstream face of the cofferdam and be connected to the slurry trench cutoff to divert flows through the outlet tunnel. The temporary impervious blanket will be removed when embankment fill placement reaches the top of the cofferdam. The cofferdam will be constructed as a permanent part of the embankment (pl. 23).

6-25 Foundation treatment will include a single line 50-foot deep grout curtain below the central portion of the core, with every other hole deepened to one-half the maximum flood pool depth. Slurry grouting of open joints and fractures and dental concrete will be used to prepare the bedrock surface for placement of fill materials.

6-26 Core material is available in sufficient quantities from the impervious borrow area approximately 2 miles southeast of the dam. No processing or special treatment is anticipated other than the control of excavation to ensure that materials with insufficient fines are not placed in the core.

6-27 Alluvial materials from the foundation excavation and downstream pervious borrow area will have to be processed through grizzleys to provide suitable gradations for the filter, upstream transition and outer shell zones. A detailed description of the anticipated processing procedure is in section 12 of Appendix A, Geotechnical.

6-28 It is anticipated that most of the spillway and drainage channel rock can be excavated by mechanical methods and that relatively little drilling and blasting will be required. Benched cut slopes will be 1V on 1H and 2V on 1H. The excavatability and compaction procedures will be investigated during the FDM study phase as part of an initial contract for a test fill. At the present time, it is anticipated that Government Canyon Ridge will be overexcavated beyond what is necessary for the drainage channel to produce borrow material for the embankment downstream transition zone.

6-29 The outlet works tunnel will be excavated by blasting in a split heading method from the downstream portal. The tunnel will be fully supported and concrete lined. Special lining and support features will be constructed at major shear crossings to accommodate the postulated 4 feet of displacement. All permanently exposed portal cut slopes in rock will be excavated to 4V on 1H and reinforced with rock bolts and pneumatically placed concrete. Although it is anticipated that the bedrock will be tight, some consolidation grouting will be conducted in the upstream portion of the tunnel.

6-30 Instrumentation of the embankment and appurtenant structures is discussed generally in Appendix A and will be addressed in detail during the FDM phase of the study.

VII. STRUCTURAL DESIGN

General

7-01 This section presents the design criteria for the structural elements of the Seven Oak Dam access roads. The structural elements for this project include a bridge, concrete slab structure, and reinforced concrete drop structures.

References

7-02 The structural elements will be designed in accordance with applicable provisions of the following:

References	Title
EM 1110-1-2101	Working Stresses for Structural Design
EM 1110-2-1806	Earthquake Design and Analysis for Corps of Engineers Project
EM 1110-2-2000	Standard Practice for Concrete
EM 1110-2-2103	Detail of Reinforced Hydraulic Structures
EM 1110-2-2502	Retaining Walls
ETL 1110-2-256	Sliding Stability
ETL 1110-2-265	Strength Design Criteria for Hydraulic Structures
AASHTO	Standard Specifications for Highway Bridges thirteenth edition, 1983

Seismic Forces

7-03 All structures will be designed to withstand the minimum earthquake acceleration of 0.05g. When earthquake forces are considered, all allowable stresses and allowable foundation pressures will be increased by 33-1/3 percent.

Unit Design Stresses

7-04 Pertinent information on unit design stresses for the structural design, in addition to those listed in the above manuals, is presented in table VII-1.

Table VII-1. Unit Design Stresses.

Concrete		
Ultimate Compressive Strength		
Cast-in-place structures other than culverts	$f'c$	= 3,000 psi
Culverts	$f'c$	= 4,000 psi
Allowable Compressive Strength		
Flexure for retaining walls and floodwalls	fc	= 0.35 $f'c$ = 1,050 psi
Ratio	n	= Es/Ec
Modulus of elasticity	Ec	= $57,000\ (f'c)^{1/2}$
Prestressed Concrete		
Compressive strength of concrete at 28 days	$f'c$	= 5,000 psi
Compressive strength of concrete at time of initial prestress	$f'ci$	= 3,500 psi
Modulus of elasticity of concrete	Ec	= 4,300 ksi
Modulus of elasticity of concrete at time of initial prestress	Eci	= 3,600 ksi

Table VII-1. Continued

Prestressing Steel			
Ultimate strength of prestressing steel	f's	=	270 ksi
Initial tensioning stress in prestressing steel	fsi	=	0.7 f's
		=	189 ksi
Effective stress in prestressing steel	fsi	=	0.8 fsi
		=	151 ksi
Reinforcing Steel (Grade 40 or 60)			
Allowable tensile strength	fs	=	20,000 psi
Modulus of Elasticity	Es	=	29,000 ksi
Design Weights			
Concrete weight	Wc	=	150 lb/ft^3
Water weight	Ww	=	62.5 lb/ft^3

(Refer to Appendix A, Geotechnical, for the weights and properties of soils.)

Bridge Structure

7-05 A new bridge will be constructed along the outlet works access road. It will have a width of about 26 feet and a span of approximately 80 feet. The bridge will be designed to be supported on two cantilever abutments. Guardrails will be provided along the bridge. Two precast, prestreesed, concrete sections will be designed for HS-20 highway loading. In addition, intermediate and end diaphragms will be provided at mid-span and at end supports respectively. In order to eliminate fixity and thereby minimize the size of the bridge pier, the span will be placed on an elastmeric bearing pad.

Drop Structure

7-06 Drop inlet structures will be provided along the roads for drainage. The vertical walls of the structures will be designed as horizontal beams without water. Earth pressure on the back of the wall

will be determined in accordance with criteria contained in civil works engineer letter "Construction stresses in retaining walls" (64-7, 22 April 1964). The lateral earth pressure would be computed for a condition of drained backfill.

VIII. RELOCATION OF STREETS, RAILROADS, AND UTILITIES

8-01 Unimpeded access to the upper canyon for SCE and other authorized personnel to operate their facilities will be maintained during project construction to minimize disturbance to existing operations. Also see Section IX, Access Roads. Under the proposed plan for the Seven Oaks Dam, there will be no permanent relocation of any public streets, highways or railroads. Detours will be required around low capacity bridges along Greenspot Road for the initial mobilization of equipment to the site. The detours will parallel the bridges and will consist of either a dip crossing or a compacted earthfill over corrugated metal culverts. The Southern California Edison Company hydropower facilities at the damsite and within the reservoir will be relocated. A water well owned by the Bear Valley Mutual Water Company (BVMWC) inside the reservoir will also require relocation. Other facilities within the canyon and borrow areas will be protected in place or avoided when possible (pls. 24 through 26).

8-02 The following paragraphs describe the relocation plans selected for the purpose of estimating a reasonable cost for the relocation of SCE facilities. The final relocation designs will be developed by the local sponsors in coordination with the utility owners.

Powerhouse

8-03 The existing SCE Powerhouse No. 2 is located approximately 1.7 miles upstream from the proposed dam and will be impacted by reservoir flood inundation as well as sediment accumulation. It is recommended that the SCE Powerhouse No. 2 be eliminated and the hydraulic head transferred to Powerhouse No. 3 downstream where new, high efficiency turbines will be installed to accommodate the additional head. The new turbines will produce the same power output as the existing Powerhouse Nos. 2 and 3 combined.

8-04 The estimated cost for the preferred plan is approximately $2,400,000, including contingencies. The cost presented is considered to be a reasonable estimate of the relocation value for these facilities. The relocation plan presented here does not obligate the local sponsor or the utility owner to implement the plan.

Flume Line

8-05 The existing SCE flume line is primarily used to convey flows through the system of three powerhouses and is also used for diversion by the San Bernardino Valley Municipal Water District. The flume line between Powerhouses Nos. 2 and 3 will be impacted by the dam and reservoir. The existing flume line in this reach consists of 7 open flume sections and 6 tunnel sections. The total length of the tunnel sections is approximately 3,100 feet and the total length of open flume sections is 9,357 feet. It is recommended that the existing flume tunnel be abandoned and replaced with a closed, pressurized conduit between Powerhouses Nos. 2 and 3 (pl. 22). The proposed conduit will be composed of a 60-inch inside diameter prestressed concrete cylinder pipe (pccp) constructed approximately 6 feet below the existing streambed. The conduit will transition into a 48-inch diameter steel pipe through the east abutment of the dam in the vicinity of the outlet works tunnel. The conduit will transition back to the 60-inch pccp downstream of the dam to connect with Powerhouse No. 3. The total length of the relocated conduit will be approximately 12,800 linear feet. The estimated cost of this plan is approximately $7,500,000, including contingencies.

8-06 Under this relocation plan, Powerhouse No. 2 will be abandoned, and the head will be transferred to Powerhouse No. 3 through the pressurized conduit. The existing flume/tunnel line will be abandoned, and the existing tunnel section through the east abutment of the dam will be sealed to prevent unregulated reservoir flow through this opening (pl. 4).

Transmission Line

8-07 An existing 66 kilovolt (kv) transmission line, consisting of 33 kv wires, is connected to the SCE system of 3 powerhouses to transmit the generated hydropower. Each powerhouse transmits at 33 kv to the total line. The line is supported on steel lattice towers from 800 to 1,700 feet apart located on the canyon side slopes approximately 100 feet above the streambed at the damsite. The transmission line will be relocated above the spillway crest elevation 2,580 feet NGVD along the east rim of the reservoir. The alignment was reviewed by SCE engineers for feasibility and cost. It is assumed that the transmission towers and lines will be installed by helicopter due to the steep terrain. The towers will be light steel lattice structures with concrete footings. The new transmission towers will be designed to prevailing standards for the 66 kv systems. The transmission line will be 33 kv wires as used in

the existing system. The towers will be inspected and maintained by foot and/or aerial patrols. The estimated cost of this plan is approximately $2,200,000, including contingencies.

Spreading Basins

8-08 The downstream pervious borrow area will be located in the water spreading basins operated by San Bernardino Valley Water Conservation District (SBVWCD). See plate 2. The preferred borrow area will impact the existing water spreading operation due to the excavation of the material for the embankment construction. Approximately 35,000 linear feet (LF) of existing dikes, various diversion and overflow gates and various corrugated metal pipes (CMP) will be impacted.

8-09 The water spreading operation will be maintained through the construction of the dam by using a phased borrow excavation plan (Appendix A, pl. A-101). Also see discussion in Appendix A, section A-12. The plan will divide the total borrow area into sectional areas where they will be excavated in sequence. Replacement dikes, diversion and drainage structures will be constructed in the excavated areas to maintain water spreading capability as the borrow excavation is phased to the next area. A new canal system will also have to be constructed to divert water into the reformed basins. Further development of this plan will require closer coordination with SBVWCD. The preliminary cost for replacing the impacted spreading basins is approximately $690,000.

8-10 An existing water well owned by East Valley Water District (EVWD) is located inside the maximum limits of the downstream pervious borrow area (pl. A-101). The well will be avoided by the borrow excavation.

Well Number 2

8-11 The existing Bear Valley Mutual Water Company (BVMWC) water well No. 2 is proposed to be relocated downstream in the vicinity of Powerhouse No. 3. The well relocation is in conjunction with the proposed elimination of Powerhouse No. 2 from the hydropower system. The final well relocation plan will be further coordinated with the San Bernardino Valley Municipal Water District (SBVMWD), as they receive credit for power produced from the pumped water.

IX. ACCESS ROADS

General

9-01 The permanent access roads are primarily designed to address the needs of the Corps of Engineers to operate and maintain the flood control structure. They will also be utilized by the United States Forest Service to manage land resources and perform fire prevention activities, and by local hydropower and water companies to operate and maintain their facilities upstream from the dam and within the reservoir. Paved all-weather access roads will be provided to the top of the dam, intake structure, spillway, and the outlet works. The roads will be paved with asphaltic concrete after the dam construction traffic has been terminated. Permanent access within the reservoir and upstream canyon will be provided by a graded, unpaved road excavated in rock and colluvium, which will join the existing unpaved access road in the streambed. Generally, turnouts will be provided approximately 500 to 1,000 feet apart on all high level permanent access roads, and metal guardrails will be provided along all permanent access roads.

Outlet Works Access

9-02 The access road to the downstream portal of the outlet works will extend off Greenspot Road approximately 1,200 feet north of the existing Greenspot Road Bridge over the Santa Ana River. The road will be a maximum of 30 feet wide with a paved width of 20 feet. The road will be paved with 2 inches of asphalt concrete on 4 inches of aggregate base course. A steel gate will be installed at the Greenspot Road entrance to prohibit unauthorized use of the road. The road will generally parallel the west bank of the Santa Ana River before crossing over to the east bank near the existing SCE Powerhouse No. 3 to continue access to the outlet works (pls. 11 through 15).

9-03 A bridge will be installed to bypass the existing steel truss bridge at the west end of the Francis Cuttle Weir across from SCE Powerhouse No. 3. The bridge will allow crossing over the Santa Ana River to the east bank under the maximum design discharge from the outlet works. The road will then generally parallel the east bank of the Santa Ana River to access the outlet works downstream portal area. The bridge will span 80 feet across the river and will be of reinforced concrete construction. Depending on its eligibility status, the existing truss bridge will be abandoned/preserved/demolished after the new bridge is installed. The existing bridge is currently being evaluated for eligibility for listing in the National Register of Historic Places. If found ineligible, the existing bridge will be abandoned and removed from the site to prevent any safety hazards.

9-04 On the left side of the stream, downstream of the outlet works portal, a parking area will be provided for post-project operation and maintenance vehicles. The parking lot will be located on a compacted fill area. The fill will consist of material that will be unsuitable for embankment construction from the Government Canyon ridge, the spillway excavation and the abutment excavation operations as well as other project required excavations.

Dam Crest Access

9-05 The access road to the crest of the dam will consist of two segments (pls. 16 and 17). The first segment consists of improving 18,350 feet of the existing unpaved Alder Creek Road off Greenspot Road by widening from its present width of approximately 16 feet to 30 feet. The second segment consists of constructing a new extension off the existing Alder Creek Road to access the top of the dam. The extension will be excavated primarily in rock to a width of 30 feet. The improved Alder Creek Road and extension will be paved with 2 inches of asphalt on 4 inches of aggregate base course for a width of 20 feet after the dam construction has been terminated. The entrance to the road extension will be gated. A sign will be placed indicating that the remainder of Alder Creek Road is unimproved so that public users are aware of the road conditions lying ahead. The top of dam will be provided with a paved service road consisting of 2 inches of asphalt concrete on 12 inches of minus 3 inch (dam embankment zone 2) or minus 9 inch (zone 4) material for a width of 20 feet. The service road will run along the top of the dam for a length of approximately 2,980 feet. See plate 6 for details.

9-06 All roadway side slope cuts in rock will be excavated to a slope of 4V on 1H. Benches will be provided for cuts higher than 20 feet. The roads can be excavated by bulldozers with rippers in most areas, with some locations requiring drilling and blasting. The excavated materials may be used for the base course and in roadway fills across gullies and ravines. However, excess materials will be hauled away and placed in designated stockpiles or disposal areas.

9-07 A number of existing storm drains and culverts across gullies and ravines along Alder Creek Road will require extensions or replacements with corrugated metal pipe culverts. Storm drains and culverts will also be installed, as necessary, along the Alder Creek Road extension. The road drainage would be designed for 50 year rainfall intensity.

Intake Structure Access

9-08 The intake structure access road (pl. 21) will extend from the road on the top of the dam and will loop around the west abutment onto the upstream face of the dam. The road will then be constructed across the upstream embankment face at about a 6 percent grade. The road will then traverse part of the east abutment until the road terminates at the intake structure at approximately elevation 2,299 feet NGVD. The road will be single laned, with turnouts and shoulders, gated at the top of the dam and paved 10 feet wide with asphalt concrete on aggregate base course or grouted stone. Gate details are shown on plate 18. The road fill section across the dam will consist of minus 12-inch (dam embankment zone 3) material placed over the upstream shell of the dam.

Spillway Access

9-09 The spillway access road (pl. 18) will extend from the road on the top of the dam and follow the reservoir rim to the spillway location. The road will join the spillway at approximately elevation 2,596 feet NGVD. The road to the spillway will be 30 feet wide and excavated in the east (left) abutment rock. The excavation may require some blasting. The road will be paved with asphalt concrete on aggregate base course for a 20-foot width. A steel gate will be provided before the spillway for safety reasons.

Canyon Access

9-10 The upstream canyon access road (pls. 19 and 20) will extend from the spillway access road to the upstream edge of the spillway, then follow the reservoir rim to an existing unpaved access road in Warm Springs Canyon where it will be gated. The road will be excavated to a width of 30 feet and will remain unpaved. Since this road will be constructed above the existing SCE open flume line, protective measures will be taken during construction to minimize possible damage to the flume line prior to its relocation. The road will enter Warm Springs Canyon at approximately elevation 2,400 feet NGVD, which is at a reservoir frequency level of 20 years at the end of project life. The existing unpaved access road in Warm Springs Canyon can be used to access the upper Santa Ana River canyon.

Geometric Design

9-11 Vehicular access roads will be designed in accordance with Department of the Army TM 5-822-2, General Provisions and Geometric Design for Roads, Streets, Walks, and Open Storage Areas. The roads will be designed as minimal all-weather low usage roads for maintenance type traffic. Design criteria used for the access roads to the top of the dam, spillway, outlet works and within the reservoir and upstream canyon will be for Class E, two-lane, mountainous roads. The access road from the top of the dam to the intake structure will be designed using criteria for a Class F, single-lane road. Due to the mountainous terrain, some of the curve radii will be shorter than that recommended in the TM. To compensate for this deviation, the recommended speed limits in these areas will be adjusted lower.

Pavement Design Values

9-12 The flexible pavement forming the paved access roads will be designed in accordance with Department of the Army TM 5-822-5. Based on test of similar types of materials, a preliminary California Bearing Ratio (CBR) value of 20 can be assigned to the subgrade when the materials are compacted to 90 percent of maximum density as determined in accordance with ASTM Method D-1557. Public use of the roads will be limited to the improved Alder Creek Road. The other roads will be accessible only to the United States Forest Service for forest management, to SCE for access to their facilities, and to the Corps of Engineers and others needed for operations, maintenance, and inspections. The average number of daily vehicle passes is estimated to be less than 25 on each lane. The traffic to which the roads will be exposed will include some small trucks and a few heavy trucks. Based on the above information, the flexible pavements will be designed in accordance with the following values:

	Access Roads	Access Road-Top of Dam to the Intake Structure
Category of Traffic	III	III
Class of Road	E	F
Design Index	3	2

9-13 A preliminary pavement section for the paved access roads will consist of a 2-inch asphaltic concrete layer on 4 inches of an aggregate base course over 6 inches of native materials compacted to 95 percent of maximum density.

Drainage Structures

9-14 Drainage structures will be provided at selected locations and at gullies and ravines where local runoffs will be collected. Excess water over the capacity of the structures will drain across the road. The sizes of the drainage structures will be determined on the basis of a 50-year frequency storm.

X. SITE RESTORATION

10-01 Plans for site restoration at the borrow areas, haul and access routes, contractor work areas, and for general clean-up were developed in coordination with local interests, Forest Service representatives, and District environmental personnel to address operational, esthetic, and environmental concerns. Primary concerns centered on impacts to existing water conservation spreading operations and water distribution systems in the vicinity of the downstream pervious borrow area and associated haul and access routes to the dam. To further reduce site impacts from the system of construction haul and access roads, the number of temporary roads will be kept to a minimum. However, some of the temporary construction roads will remain and be incorporated into the post-project permanent access road system. All other temporary roads will be regraded and seeded to return to natural vegetation. The recommended mitigation plan calls for riparian revegetation on 60 acres of land between Greenspot Road and the Seven Oaks damsite along the Santa Ana River, exclusive of the permanent access roads and the dam tender's site. This would be implemented after the dam construction is completed.

10-02 Haul routes and contractor work areas were located to cause minimum damage to the terrain and to preserve, within practical limits, the natural state of the project area. Clearing and grading will be stipulated for all required contractor work areas. Erosion control measures will be provided for all existing improved and relocated roads within the project area, as well as for the dam embankment itself.

XI. RECREATION

11-01 The recreation plan proposed for the Seven Oaks Dam was presented in the Supplement to the Phase I GDM and included trails, picnic areas and two intensive use areas. The plan had been developed as a part of an intensive cooperative planning effort conducted under the terms of the 13 August 1964 Memorandum of Agreement (MOA) between the Secretary of the Army and the Secretary of Agriculture. The MOA states that "planning will be pointed toward achieving the maximum public benefits from each project," compatible with the primary purpose of the water storage facility. The Phase I Supplement's recommended plan proposed development of low density recreation facilities on project lands that will result in a positive benefit-to-cost ratio. The U.S. Forest Service furthermore requested consideration of "in lieu site" recreation development at other areas within the national forest that were more accessible than the Seven Oaks site. Further study of "in lieu site" recreation development was not undertaken because it was not consistent with Corps of Engineers policy regarding recreation development.

11-02 The U.S. Forest Service (USFS) has recently completed a draft San Bernardino National Forest Land and Resource Management Plan. The recreation plan presented in the Phase I Supplemental GDM was not consistent with this document. The Forest Service, in a letter dated 13 October 1987 to the Corps of Engineers, requested that the recreation plan proposed for Seven Oaks Dam be consistent with their Land and Resource Management plan. Due to the inconsistency of the two plans, further studies on recreational development at Seven Oaks Dam will not be considered at this time. In the future, if there are any changes in the design operation and maintenance, especially in the event of a water conservation pool being developed behind the dam, a re-evaluation of National Forest Management objectives within the project area will be undertaken.

11-03 The existing recreation setting of Seven Oaks is a relatively pristine natural area located within the San Bernardino National Forest. Recreational use of the canyon in the past has been low to moderate due to its restricted access. The site offers opportunities for hunting, fishing, hiking, picnicking, backpacking and equestrian

uses. The USFS Land and Resource Management plan prescribes an emphasis on "dispersed" recreation opportunities that will be compatible with the proposed design of an earth and rock filled dam with a seasonally operated debris pool. USFS prefers development with no hard structures such as picnic facilities and restrooms. The USFS plan emphasizes an ideal "natural" or "wilderness" recreation, acessible by foot traffic only. This bare minimum accessible trail will lead the public to hiking, fishing, and camping opportunities. See Appendix D for the Seven Oaks Dam Resource Use Master Plan.

XII. ENVIRONMENTAL ANALYSIS

Environmental Impacts

SEDIMENTATION

12-01 Seven Oaks Dam has a sediment storage allocation of 32,000 acre-feet of sediment during project life. As a result of detailed sediment distribution studies in the Phase II, it was estimated that a much greater depth of accumulation would occur than previously reported in the Phase I Supplemental GDM. The estimated sediment at the intake after 100 years was increased from 80 feet to 165 feet deep. While the depth of sediment accumulation has increased, the physical configuration of the canyon has limited the areal extent of the increased sediment distribution.

12-02 Construction-related impacts, such as construction of haul and access roads, and borrow areas, will liberate sediment from exposed soil surfaces. The abutment excavation will also expose material to the weathering process. Increased turbidity to surface water is expected, with fines and silts from loosely-consolidated sub-surface materials contributing the most. It is expected that heavy rains after project construction begins will carry excavated debris and other fine particles into water courses, which will result in increased turbidity. Increased erosion will also result on newly-exposed materials.

12-03 The presence of Seven Oaks Dam will significantly reduce the amount of sediment moving downstream. As described in the 1985 Phase I Supplemental GDM, Supplemental Environmental Impact Statement (SEIS) (pp. 75-76), the character of the outwash plain below the dam will change, although tributaries downstream of the dam will continue to transport and deposit sediment. Impacts resulting from sediment accumulation behind the dam, as well as the reduction in sediment downstream, have been addressed in the 1985 Phase I SEIS.

WATER RESOURCES

Hydrology and Water Use

12-04 Impacts to hydrology and water use have not changed significantly from the 1985 Phase I SEIS.

Water Quality

12-05 Impacts to water quality have changed since the Phase I SEIS. The U.S. Forest Service will not be developing recreation as previously planned in the Phase I. Only "non-developed" recreation will be allowed in the project area. Therefore, recreational use is not expected to increase significantly, and coliform counts and turbidity will not be as great as expected.

12-06 Impacts to water quality are not expected to be significant. However, it is possible that organic material and nutrients would cause algae blooms and anoxic conditions upon initial reservoir filling, particularly if vegetation is not removed. The potential for significant impacts to occur does exist, however, if any construction-related fluids should escape containment areas. Considerable construction-related turbidity is expected for relatively short duration during runoff events. The turbidity may appear significant in the Santa Ana River, but it is doubtful that the impacts from it would be significant. Similiar turbidity appears naturally during the initial storm events of the season as natural accumulations of sediment from weathered rocks and dust are washed into the watercourses. A beneficial impact is more recharge to groundwater of high quality runoff stored in the debris pool.

AIR QUALITY

12-07 Impacts to air quality have not changed significantly from the 1985 Phase I SEIS.

LAND USE AND SOCIAL CONCERNS

Important Farmlands

12-08 No farmlands would be impacted at the reservoir site. The Mill Creek borrow site contains important farmlands which will be impacted. Since the borrow site is proposed for urban development, however, the project will not create impacts to important farmlands that are not already projected.

Recreation

12-09 Impacts to existing recreational opportunities due to construction activities and structural improvements have been addressed in the 1985 Phase I SEIS. Since the Phase I SEIS, the U.S. Forest Service has withdrawn their request for "in lieu" recreational development in the National Forest. In keeping with the wilderness character of the San Bernardino National Forest, developed features (picnic areas, restrooms, etc.) will not be included in the overall recreation plan.

Growth Inducement

12-10 Growth inducement as a result of the construction of Seven Oaks Dam has been addressed in the 1985 Phase I SEIS.

TRANSPORTATION AND UTILITIES

Utilities

12-11 The U.S. Forest Service access road will be relocated to maintain continued access for USFS and SCE personnel to manage and operate their facilities. The Southern California Edison transmission line and flume system and the Bear Valley Mutual Water Company well will be relocated due to reservoir floods and sediment accumulation that will impact them. The Southern California Edison Powerhouse #2 will also be impacted by floods and accumulating sediment.

Access

12-12 Impacts are expected to occur to the existing Alder Creek Road. These impacts have been addressed in the 1985 Phase I SEIS.

Transport of Borrow Materials

12-13 Borrow material will be transported to the damsite from the downstream pervious borrow area and the impervious borrow area, via temporary constructed haul routes. Some impact to the existing Greenspot Road and the USFS/SCE access road is expected, in terms of usage and heavy construction vehicle cross traffic due to intersections with haul routes (pls. 24 through 26).

NOISE

12-14 The Seven Oaks Dam area is a relatively undisturbed area, with little human-induced noise. The project will have short-term impacts to the environment, as construction-related noise will be present.

BIOLOGICAL RESOURCES

12-15 The 1985 Phase I SEIS addressed impacts to biological resources, with the exception of borrow areas, haul and access routes, disposal areas and any changed conditions. In total, 534 acres of upland habitat

will be impacted in the vicinity of the dam (only 127 acres will be permanently lost). Twenty-six acres of riparian habitat and 4.6 miles of aquatic stream habitat will also be lost. Between 350 and 764 acres of alluvial scrub will be lost due to the excavation of the pervious borrow site. About 1,000 acres could be excavated at the impervious borrow site of which 70 percent is vegetated with Chaparral and Coastal Sage Scrub.

12-16 Eriastrum and Centrostegia, two plant species, have been listed on the Federal Endangered Species list as of September 1987. The construction of Seven Oaks Dam will impact Eriastrum. The project will not impact Centrostegia. Direct impacts to Eriastrum will come about as a result of the dam construction, and the excavation of the pervious borrow area, which is just downstream of the dam. Additional impacts will result from the reduction of floodflows once the dam is in place. The Eriastrum is dependant upon the deposition of fresh sand from flood flows in order to disperse and outcompete other plant species.

CULTURAL RESOURCES

12-17 The proposed Seven Oaks Dam will be comprised of a number of features, the construction of which will have the potential for directly impacting cultural resources. These features include the dam embankment, levels of reservoir inundation, access and haul roads, parking lots and borrow areas. The construction activities will impact three prehistoric sites, and 36 historic sites. Two of the historic sites (but none of the prehistoric sites) are eligible for inclusion in the National Register of Historic Places (NRHP). Seventeen of the historic sites are not considered eligible for listing on the NRHP. The remaining sites have yet to be determined for eligibility.

Mitigation

BIOLOGICAL RESOURCES

12-18 The 9 July 1987 Chief of Engineers Report provided guidance in respect to the mitigation package that was developed for the 1985 document (acquisition of Filaree Flats, Section 5 and replacement habitat on Santa Ana Wash and Bear Creek). The Report stated that impacts to fish and wildlife resources were overstated and that the least-cost efficient component of the approved mitigation package be modified or deleted. Filaree Flats was specifically identified for inclusion in the mitigation plan.

12-19 Approximately 60 acres of relatively level land in the vicinity of the streambed, just downstream from the dam and upstream from Greenspot Road, have been identified for development of riparian habitat. These 60 acres are expected to have sufficient quantities of groundwater close enough to the surface to maintain riparian vegetation after 2 years of irrigation support. Mitigation improvements needed to develop this riparian habitat will include site preparation, plant material, and a temporary irrigation system.

12-20 The endangered species Section 7 consultation for the Eriastrum between the USFWS and the COE has not been formally concluded. However, informal agreement was reached shortly before completion of this final Phase II GDM. The COE and USFWS agree that approximately 700 acres of the Santa Ana River Wash between Greenspot Road and the Norton Air Force Base are necessary to ensure that the Eriastrum can persist into the future. Areas targeted for acquisition are illustrated in the Main Report and SEIS. The detailed rationale for this mitigation is presented in the SEIS.

CULTURAL RESOURCES

12-21 During the Phase II studies, a Memorandum of Agreement (MOA) will be signed with the Advisory Council on Historic Preservation and the State Historic Preservation Officer that will ensure that the Corps will mitigate for impacts to significant cultural resources in the area of potential effect.

XIII. DIVERSION AND CONTROL OF WATER DURING CONSTRUCTION

13-01 The diversion and control plan for Seven Oaks Dam presented herein takes into consideration each of the three outlet works control alternatives: downstream control, upstream control, and mid-tunnel control. The outlet works design with downstream control uses an 18-foot-diameter, concrete-lined tunnel for diversion during the initial construction of the dam. As the embankment fill construction rises in elevation, an 11-foot diameter steel penstock will be installed within the concrete-lined tunnel for subsequent diversion use and for the operational requirements of the dam. The outlet works design with upstream control uses an 18-foot-diameter, concrete-lined tunnel with an upper access gallery for diversion during the construction of the dam. This same tunnel will be used for the operational requirements of the dam. For the mid-tunnel control alternative, the outlet works design uses an 18-foot-diameter, concrete-lined tunnel for diversion during the construction as well as for the subsequent operational requirements of the dam. See also discussion in Section V, Hydraulic Design.

Cofferdam

13-02 An evaluation of impacts from dam breach overflows for various cofferdam heights was made to determine two threshold cofferdam heights. The first represented the height at which the overflow resulting from a breach during initial overtopping will not induce any damages downstream, particularly to SCE powerhouse No. 3 located about one mile downstream from the project site; the second represented the height at which the overflow resulting from a breach during initial overtopping will induce damage at SCE Powerhouse No. 3, but not to any downstream development. The heights were determined to be 50 feet and 75 feet, respectively.

13-03 The two threshold cofferdam heights were then evaluated for the diversion capacity afforded when combined with an 18-foot-diameter outlet tunnel. Flood routings indicated that the 50-foot-high cofferdam will provide diversion capacity up to a 18-year flood event and that a 75-foot-high cofferdam will provide diversion capacity up to a 26-year flood event (fig. 5). A diversion capacity for between 10- and 15-year flood events is in the realm of normal practice for the purpose of protecting the below streambed work area during foundation excavation, foundation grouting and below streambed fill placement. The below

streambed work will require approximately a 13-month period to be completed. Therefore, based on the adequate diversion capacity, lower cost for construction, and the virtual elimination of the risk to SCE Powerhouse No. 3 and downstream development, the 50-foot-high cofferdam was selected.

13-04 The cofferdam will be constructed with a height of 50 feet. The cofferdam will be constructed across Government Canyon with the right (west) end tied into the right abutment rock mass of the main embankment and the left (east) end tied into Government Canyon ridge (pl. 23). The cofferdam will be continued across the Santa Ana River with the right end tied into Government Canyon ridge and the left end tied into the left abutment rock mass of the main embankment. The cofferdam will be constructed along the upstream toe of the main embankment (pl. 23).

13-05 The existing streambed elevation in Government Canyon is approximately 2,125 feet NGVD. The top elevation of the Government Canyon portion of the cofferdam will be at 2,175 feet NGVD. The existing streambed elevation in the Santa Ana River is approximately at 2,100 feet NGVD. The top elevation of the Santa Ana River portion of the cofferdam will be at 2,150 feet NGVD.

13-06 The cofferdam will be constructed from processed alluvium excavated from the foundation of the main embankment. In addition, approximately 800,000 cubic yards of material will be excavated from Government Canyon ridge to form a drainage channel connecting Government Canyon with the Santa Ana River. The purpose of this drainage channel is to allow flows from Government Canyon to reach the diversion tunnel.

Outlet Works

13-07 Diversion and control of water during construction of the outlet tunnel would be minimal due to the offset location of the inlet portal from the main river channel. Low dikes may be constructed at the portals to prevent damages from a 10-year flood event. Construction of the tunnel will likely proceed from the downstream end and the tunnel will not be completely open until the summer months, when the flood threat is minimal. The approach channel will be excavated prior to the below streambed construction. A general plan and details of the outlet works are presented in Appendix B. The pre-formed plunge pool energy dissipator will be constructed prior to diversion of the river.

Main Embankment

13-08 Construction on the embankment will be scheduled to begin in April of main embankment construction year 1 with the mobilization of equipment to the site (pl. 30). Summer season diversion efforts through the 18-foot diameter outlet tunnel will begin with the excavation of the foundation and approach channel, installation of the slurry trench cutoff, dewatering wells, and construction of the cofferdam in May of year 1. The 50-foot-high cofferdam is expected to be completed in three months along with the slurry trench, excavation of the approach channel

to the outlet tunnel, and installation of dewatering wells. Excavation of the foundation, approximately 2.5 million cubic yards ($myds^3$) of material, is expected to take the next 4 to 5 months until the end of September. Summer diversion requirements are expected to be minimal, with the Southern California Edison Company (SCE) diverting most of the streamflows into their aqueduct and around the damsite. Therefore, summer diversion will consist mostly of groundwater flow brought to the surface by the slurry trench. These flows will be diverted through the tunnel. Grouting of the foundation is expected to take 4 months until the end of January (year 2). Approximately 4.5 $myds^3$ of embankment fill will then be placed below streambed to the tunnel inlet invert elevation 2,100 feet NGVD. The placement of embankment fill below streambed is expected to take 6 months until the end of July. As shown on plate 30, the construction activities below streambed, including the cofferdam, will require 14 months, starting in May of year 1 and ending as early as the end of the following July of year 2. Therefore, the 50-foot-high cofferdam and the 18-foot diameter diversion tunnel will offer protection to the below streambed work for a 18-year flood event for one rainy season from November to March. Refer to figure 5.

13-09 The first stage of above streambed construction on the dam will be to raise the embankment level with the top of the 50-foot-high cofferdam (El. 2,150 ft. NGVD). See plate 31. This will require placement of approximately 4 $myds^3$ of embankment fill and is expected to take 4 months. Therefore, the first stage of embankment construction (to El. 2150 ft. NGVD) can be accomplished as early as the end of November (year 2). The cofferdam will provide protection to the fill placed and to the placement activities to this level. Failure of the cofferdam by overtopping and breach formation, although unlikely, will cause damage to the fill placed; however, no damage will be induced to downstream development (see Emergency Action Plan During Construction inundation mapping, Appendix C).

13-10 The second stage of construction on the dam will be to raise the embankment to attain 100-year overtopping protection, El. 2,310 ft. NGVD for downstream and mid-tunnel outlet works control (El. 2.317 ft. NGVD for upstream outlet works control). This elevation will provide protection greater than the maximum flood of record at the site. Nevertheless, downstream communities will be subject to risk during one winter season with less than SPF overtopping protection. Construction of the embankment will continue beyond elevation 2,310 NGVD (2,317 NGVD upstream control) through the winter season. Assuming a finish in November (year 2) for the first stage of construction to elevation 2,150 feet NGVD, construction of the dam to the 100-year level will be completed about 13 months later at the end of December (year 3). Fill placement will average 1.1 $myds^3$ per month to place the 13.6 $myds^3$ (14.2 $myds^3$) required to reach elevation 2,310 feet NGVD (2,317 feet NGVD upstream control).

13-11 The third stage of construction on the dam will be to raise the embankment to attain SPF overtopping protection, El. 2,375 feet NGVD for downstream and mid-tunnel outlet works control (El. 2,384 feet NGVD for upstream outlet works control) with the 18-foot diameter diversion

tunnel. This will require placement of 5.4 myds^3 (5.5 myds^3) of fill and is expected to take 5 months from January (year 4) to the end of May (year 4). With the downstream control alternative, concurrent construction on the 11-foot-diameter steel penstock within the tunnel can take place during the dry season from April to October (year 4). In this case, the embankment must be constructed up to elevation 2,476 feet NGVD before the winter season begins in November (year 4) to maintain SPF overtopping protection due to the reduced diversion capacity of the smaller steel penstock. This will require placement of an additional 7.2 myds^3 (6.6 myds^3) of embankment and will take 7 months, but can be completed by the end of December (year 4). The gate transitions, gate mechanisms, and intake tower will be installed the following dry season beginning in April of year 5 as the embankment is topped out. The final 5 myds^3 of embankment placement is expected to take 7 months due to reduced production rates as the working area on the fill becomes restricted. Therefore, the dam will be topped out by the end of July (year 5). The embankment placement schedule is shown on plate 31.

Flood Emergency Plans

13-12 Under this construction schedule, downstream communities will be at risk of substantial induced flooding during a single winter season from November to March (year 3). Flooding would occur in the event the partially constructed dam is overtopped and breached by an event greater than the 100-year flood. To address the remote possibility of overtopping flood events creating downstream hazards, an Emergency Action Plan During Construction has been prepared in draft form and included as appendix C to this volume. The plan includes inundation mapping for the areas downstream from the dam depicting areas which would be flooded in the event a hypothetical breach occurred during initial overtopping of the main embankment during construction. The flooding is depicted for a hypothetical breach when the main embankment is at a height of 100, 200, 300, and 400 feet in height.

13-13 Inundation maps depicting areas flooded in the event a hypothetical breach occurred during initial overtopping of the cofferdam are also shown in the Emergency Action Plan. Since the cofferdam height was established at the threshold level at which no damage will occur downstream in the event of a breach, there will be no need to evacuate downstream communities; however, people will be warned to stay out of the river. Therefore, a plan to notify local emergency service officials is identified for such an event. No coordination of the Emergency Action plan has been accomplished, coordination will be scheduled to be accomplished prior to embankment construction.

13-14 The plan establishes the events that will trigger the notification of inspection teams, notification of key personnel within the Los Angeles District and South Pacific Division Offices, inspection procedures and the responsibilities of each person involved. The names and phone numbers of local emergency service officials will be included for notification in the event an evacuation of downstream areas is necessary.

13-15 The plan will be provided to the Resident Office staff, all personnel on the notification lists, the Reservoir Operation Center staff, and to local emergency planning officials. The local emergency planning officials will be responsible for developing and implementing evacuation plans. A meeting will be held with local officials to provide guidance on how the inundation maps may be used in evacuation planning and to answer any questions regarding the action plan. The construction contractor will also be provided copies of the report and will be briefed prior to construction and each flood season on the responsibilities of key personnel.

XIV. REAL ESTATE REQUIREMENTS

14-01 The Seven Oaks Dam is located approximately 8 miles northeast of the City of Redlands and within the boundaries of the San Bernardino National Forest.

14-02 The damsite is in the upper Santa Ana River canyon in rugged mountainous terrain. The dam will be an earth-rockfill structure 550 feet high with a crest length of 2,980 feet, 40 feet wide at the top and approximately 2,000 feet wide at the base. The spillway is unlined and will be excavated in a saddle east of the dam. The damsite requires approximately 125 acres of private land and will be purchased in fee. Mitigation lands include 649 acres at section 5 near the dam, 137.52 acres in Filaree Flat, 60 acres in the streambed between Greenspot Road and the dam, and 700 acres in the Santa Ana Wash between Norton Air Force Base and Greenspot Road. An additional one acre parcel will be required in the area between the dam and Greenspot Road for a permanent administration building. This acre will be purchased in fee. The reservoir will cover approximately 800 acres, nearly all within the San Bernardino National Forest and have a capacity of 145,600 acre-feet.

14-03 The material to be used for the construction of the dam will come from two major borrow areas. The borrow areas will be acquired as an easement for a 10-year period. The total number of acres within the maximum limits of the downstream pervious borrow area is 764 acres of which 631.66 is privately owned, with the balance being Government lands owned by the Bureau of Land Management. See plate 2. The area within the maximum limits of the downstream pervious borrow is a conservative estimate which provides for more land than is required. To determine the real estate cost, a more reasonable area is estimated that will provide an adequate volume of material for embankment construction and which is strategically located for construction and geotechnical reasons. Note preferred area boundaries on plate 2. Based on these consideration, approximately 415 acres will be required of which 325 acres are privately owned and 90 acres are Government lands.

14-04 The downstream pervious borrow area will be located in the water spreading basins operated by San Bernardino Valley Water Conservation District (SBVWCD). The water spreading operation will be maintained during the embankment construction by using a phased borrow excavation

plan as shown in Appendix A, plate A-101. According to the plan, the total borrow area will be divided into sectional areas and excavated in sequence. Dike and diversion structures will be replaced in the excavated area to maintain water spreading capability as the borrow excavation is phased into the next area. The borrow area will be excavated to an average depth of 40 feet (pl. A-55). The cost to remove and replace the SBVWCD water spreading facilities will be part of the relocation cost not included in the real estate estimate. See Section VIII.

14-05 The impervious borrow area will also require a 10-year easement for the purpose of removing materials for the construction of the dam. The area within the maximum limits of the impervious borrow is mainly in private ownership, the largest property being Sunrise Ranch, of which 814 acres are to be acquired in a 10-year easement. The ranch is zoned Planned Unit Development with the owner having an approved subdivision plan. In addition to the Sunrise Ranch, approximately 100 acres in three tracts are required for a total of 914 acres to be taken in a 10-year easement for the impervious borrow area. One hundred seventy acres of U.S. Forest Service land will also be used for this purpose. Based on the same reasons as discussed in paragraph 14-03, the more reasonable land estimate for cost purposes requires approximately 640 acres of privately owned land only. Note preferred area boundaries on plate 2.

14-06 The impervious borrow area will be more gently sloping in hilly and ravine areas after the removal of materials by the borrow operations at the end of the 10-year easement. The borrow area will be excavated to an average depth of 15 feet. Two existing waterlines, the Greenspot pipeline owned by the San Bernardino Valley Municipal Water District (SBVMWD) and the Bear Valley Highline owned by the Bear Valley Mutual Water Company, are located within the impervious borrow area. These utilities will be avoided or protected in-place during the borrow operations.

14-07 The perpetual road easement covers the existing U.S. Forest Service road right-of-way and will be increased in width due to the proposed road improvements for project access in the rough terrain. Two private ownerships will be affected requiring easement over approximately 37 acres.

14-08 The perpetual flowage easement covers the reservoir and the area downstream of the spillway. It encumbers approximately 953 total acres, 892 acres being Government land owned by the U.S. Forest Service, the balance 61 acres in three private ownerships.

14-09 The damsite, section 5, Filaree Flat, flowage and road easements that are privately owned are zoned M-1 (limited manufacturing). The downstream pervious borrow area is in the floodplain and is zoned FP-1 and FP-2. The impervious borrow area is zoned PUD (Planned Unit Development).

14-10 There are two occupied dwellings within the impervious borrow area which will be removed or razed in the process of taking the borrow material to the damsite.

14-11 There is a two-grave cemetery at the damsite. The cemetery is located just below the downstream portal of the outlet works tunnel. It has been determined that the cemetery has no historical significance nor is it considered a cultural resource. The local sponsors are responsible for relocating the graves at no cost to the Government.

14-12 Approximately 2,736 acres of private land and 982 acres of Government land will be required for the project. The 892 acres of Government-owned land necessary for the dam, spillway, and reservoir is under the jurisdiction of the U.S. Forest Service. The Government-owned land required for the construction of the dam will be accomplished by the transfer of accountability from the Forest Service to the U.S. Army Corps of Engineers. All necessary lands for the flood control reservoir will be acquired by a special use permit from the Forest Service without any transfer of accountability. The 90 acres of Government-owned land in the pervious borrow area will be acquired as an easement for a 10-year period. A special use permit will be acquired from the Bureau of Land Management without any transfer of accountability. The approximate 982 acres of Government-owned land will be acquired at no cost to the project.

The estimated real estate costs of Seven Oaks Dam is:

Damsite	125 acres	$ 75,000
Administrative office area	1 acre	2,000
Impervious borrow area (10-year easement)	640 acres	2,560,000
Downstream pervious borrow area (10-year easement)	325 acres	260,000
Perpetual road easement	37 acres	11,000
Perpetual flowage easement	61 acres	30,000
Mitigation lands (Section 5, west of dam)	649 acres	390,000
Mitigation lands (Filaree Flats)	138 acres	69,000
Mitigation lands (streambed below dam)	60 acres	120,000
Mitigation lands (Santa Ana Wash)	700 acres	2,450,000
Subtotal	2,736 acres	5,967,000
Contingencies (20%)		1,193,000
Relocations (PL 91-646)		50,000
Administrative		248,000
Total Costs		$7,458,000

XV. FLOODWAY DELINEATION

15-01 A preliminary floodway has been delineated for the 100-year flood with project and is shown on plates 34 and 35. The delineation is based upon reconnaissance-level floodplain hydraulic computations. A detailed evaluation of the 100-year and 500-year floodways is presently scheduled as a separate feature design memorandum. The detailed floodplain will be delineated at that time.

15-02 The floodway will be evaluated in accordance with the Federal Emergency Management Agency (FEMA) guidelines, except that flows under future conditions will be utilized with Seven Oaks Dam in place and operational. The local sponsors will have the responsibility to manage the floodway and fringe floodplain in accordance with regulations established by FEMA.

XVI. COST ESTIMATES

16-01 The cost estimates are based on unit price data from recent bids for similar items of construction on other projects and on unit prices derived using established estimating procedures. Quantities of principal construction items were estimated based on detailed designs. In accordance with EM 1110-2-1301, a 15 percent contingency is added to the estimated construction cost.

First Cost

16-02 The first cost of the proposed Seven Oaks Dam is presently estimated at $316,283,000 and summarized in table XVI-1. A detailed estimate of the first cost is shown in table XVI-2. The total, flood control first cost, includes the construction of the embankment, spillway, outlet works, associated access roads, administration facilities, hydrologic facilities and communication equipment. Also included are costs for contingencies, engineering and design, supervision and administration, an operation and maintenance manual, land acquisitions and easements, relocation of impacted facilities, and biological and cultural mitigations.

16-03 Unit prices for the construction items are derived from a breakdown of plant, labor and material, and include contractor markups for overhead, bond and profit. The estimated prices are based on the following assumptions and considerations:

a. The care and diversion of water cost consists of a plan that provides a slurry trench cutoff to bedrock, a temporary impervious blanket on the upstream face of the cofferdam which ties into the slurry trench, and dewatering wells.

b. The test fill/excavation is estimated to be the first construction start for the Seven Oaks Dam element and will include test excavations, test fills and a streambed access road from the existing Greenspot Road to the damsite. This road will subsequently be used as a haul road from the downstream pervious borrow area and the impervious borrow area. Also see discussion in Appendix A, Geotechnical, Section A-XIII.

c. The abutment excavation is estimated to include the removal of abutment colluvium and rock to provide suitable abutment slopes for embankment fill placement. The cost for the construction of heavy equipment access roads to the top of both abutments, which will later be incorporated into the permanent access road system is separated. See section IX for more access road details. The abutment excavation cost does not include the initial construction of the access roads. This cost and the costs of the final asphalt concrete pavement, guardrails and other road items are included under cost account no. 8.0, Roads.

d. The cost of the cofferdam is based on the concurrent excavation of the foundation alluvial material, processing this material through a "grizzley" or screen to obtain minus 12-inch material, and spreading and compacting the material to construct the cofferdam. Since the cofferdam material is obtained from excavations within the main embankment foundation, the total required foundation excavation volume is decreased by the volume utilized in the cofferdam. The remaining required foundation excavation volume will be excavated and hauled to the downstream pervious borrow area for stockpiling. The cost of processing the material through "grizzlies" and transporting it back via belt system for the construction of the embankment is included in the pervious borrow (below streambed) item.

e. The cost of the pervious material from the downstream pervious borrow area includes excavation, processing of the material through "grizzlies" to separate the minus 12-inch and the minus 3-inch materials, and transporting the materials to the damsite by using a combination of a belt system and truck hauls. The depth of excavation will range from 30 to 40 feet. Also see discussion in Appendix A, section A-XII.

f. The cost of the impervious borrow material is based on truck hauls from the borrow area. A belt system will transport the material up the face of the embankment for placement above streambed. The material will be pre-wet in the borrow area.

g. The costs for the spillway and Government Canyon Ridge excavations are based on excavatability with conventional mechanical equipment. It is assumed that processing will not be required for the materials to be used in the embankment. Truck hauls will be used to transport the Government Canyon Ridge material to construct the lower elevations of the embankment. The material from the spillway will be transported by a belt system to construct the upper elevations of the embankment.

h. The cost for the outlet works is based on the design and cost estimate presented in Appendix B by the Portland District Corps of Engineers.

i. The relocation cost includes the relocations of SCE and BVMWC facilities, a gravesite, and the San Bernardino Valley Water Conservation District's (SBVWCD) water spreading facilities. The SBVWCD diversion gates, drainage structures, dikes and ponds will be replaced in conjunction with a sequenced borrow operation. See Appendix A, section A-XII.

Table XVI-1. Summary of First Cost.
(October 1987 Price Levels)

Acct No.	Description	Amount
	CONSTRUCTION	
03.	Reservoir	$ 67,000
04.	Dam	250,645,000
08.	Roads	3,956,000
18.0	Cultural resource preservation	200,000
19.0	Buildings, ground and utilities	115,000
20.0	Permanent operating equipment	963,000
30.0	Engineering and design	15,425,000
31.0	Supervision and administration	15,357,000
51.22	Operation and maintenance manual	100,000
	Total, construction	$286,828,000
	LANDS AND DAMAGES	
	Lands	3,824,000
	Mitigation lands	3,634,000
	RELOCATIONS	12,990,000
	Total, lands, damages, and relocations	$ 20,448,000
	Preconstruction engineering and design	$ 9,007,000
	TOTAL, FLOOD CONTROL FIRST COST	$316,283,000

Table XVI-2. Detailed Estimate of First Cost.
(October 1987 Price Levels)

Acct. No.	Description	Quantity	Unit	Unit Price	Subtotal
03.	Reservoir clearing & grubbing (including contingencies)	1	Job	L.S.	$ 67,000
04.	Dam				
0.1	Main Embankment				
	Care & diversion of water	1	Job	L.S.	$ 2,401,000
	Test fill/excavation	1	Job	L.S.	3,970,000
	Abutment excavation	1,256,000	CY	L.S.	5,132,000
	Clearing & grubbing				
	Embankment	130	Acre	$ 2,000	260,000
	Borrow areas	580	Acre	$ 1,000	580,000
	Borrow				
	Impervious (core)	4,623,000	CY	$ 4.00	18,492,000
	Pervious (Filter, Transition, and Shell)	17,719,000	CY	$ 4.25	75,306,000
	Pervious (Below streambed)	1,742,000	CY	$ 3.90	6,794,000
	Excavation				
	Alluvial foundation	1,742,000	CY	$ 3.40	5,923,000
	Rock foundation	139,000	CY	$ 4.50	626,000
	Gov't. Canyon drainage channel (incl. 70,000 cy of stockpiled test excavation)	4,308,000	CY	$ 2.70	11,632,000
	Foundation drill & grout				
	Mob. and demob.	1	Job	L.S.	100,000
	Drilling grout holes	45,000	LF	$ 10.00	450,000
	Grout connections	1,000	Ea	$ 50.00	50,000
	Placing grout	4,500	Sa	$ 30.00	135,000
	Pressure testing	500	Hr	$100.00	50,000
	Drill set-ups	900	Ea	$ 50.00	45,000
	Drilling exploratory holes	1,500	LF	$ 68.00	102,000
	Cofferdam (u/s), including excavation, processing and compaction	544,000	CY	$ 4.20	2,285,000
	Embankment				
	Impervious	4,623,000	CY	$ 0.70	3,236,000
	Filter	2,656,000	CY	$ 0.65	1,726,000
	Transition (u/s)	10,601,000	CY	$ 0.65	6,891,000
	Transition (d/s)				
	Gov't Canyon material	4,706,000	CY	$ 0.30	1,412,000
	Spillway material	8,449,000	CY	$ 0.65	5,492,000

Table XVI-2. (Continued)

Acct. No.	Description	Quantity	Unit	Unit Price	Subtotal
	Shell	6,910,000	CY	$ 0.65	4,492,000
	Road on u/s face	27,000	CY	$ 0.65	18,000
	Foundation and Abutment treatment (Cleaning, scaling, slurry grouting & dental concrete)	1	Job	L.S.	500,000
	Access roads				
	Top of dam	1	Job	L.S.	232,000
	Intake structure	1	Job	L.S.	476,000
	Site restoration esthetic treatment	1	Job	L.S.	284,000
	Biological mitigation improvements (including contingencies)	1	Job	L.S.	1,300,000
	Subtotal				$160,392,000
	Contingencies (15%)				23,864,000
	Subtotal, main embankment				$184,256,000
0.2	Spillway				
	Excavation				
	Rock	7,571,000	CY	$ 4.00	$ 30,284,000
	Control sill	1,600	CY	$ 30.00	48,000
	Concrete, sill	1,600	CY	$100.00	160,000
	Cement, portland	9,100	CWT	$ 4.65	42,000
	Reinforcing steel	20	Ton	$800.00	16,000
	Subtotal				$ 30,550,000
	Contingencies (15%)				4,583,000
	Subtotal, spillway				$ 35,133,000
0.3	Outlet works				
	Reference Appendix B for Estimate, Downstream Control				$ 27,179,000
	Subtotal, (not including access road)				$ 27,179,000
	Contingencies (15%)				4,077,000
	Subtotal, outlet works				$ 31,256,000
	Total, dam				$250,645,000

Table XVI-2. (Continued)

Acct. No.	Description	Quantity	Unit	Unit Price	Subtotal
8.0	Roads				
	Outlet Works	1	Job	L.S.	$ 985,000
	Alder Creek improvement	1	Job	L.S.	1,175,000
	Alder Creek Extension	1	Job	L.S.	415,000
	Spillway Access	1	Job	L.S.	213,000
	Warm Springs Canyon	1	Job	L.S.	652,000
	Subtotal, roads				$ 3,440,000
	Contingencies (15%)				516,000
	Total, roads				$ 3,956,000
18.0	Cultural resource preserv. (including contingencies)	1	Job	L.S.	$ 200,000
19.0	Buildings, grounds & utilities				
	Administration facilities	1	Job	L.S.	$ 100,000
	Subtotal, buildings, grounds & utilities				$ 100,000
	Contingencies (15%)				15,000
	Total, buildings, grounds & utilities				$ 115,000
20.0	Permanent operating equipment				
	Hydrologic facilities				
	Reservoir staff gauges	1	Job	L.S.	$ 4,800
	Instrument house	1	Job	L.S.	4,000
	Precipitation and water surface recording facilities	1	Job	L.S.	57,700
	Downstream gauging station facilities	1	Job	L.S.	16,700
	Gate Recorders	1	Job	L.S.	3,800
	Sediment ranges (monumented)	1	Job	L.S.	50,000
	Communications equipment				
	Repeater station	1	Job	L.S.	100,000
	Geotechnical instrumentation	1	Job	L.S.	600,000
	Subtotal, permanent operating equipment				$ 837,000
	Contingencies (15%)				126,000
	Total, permanent operating equipment				$ 963,000
	Total, reserv., dam, roads, bldgs. & permanent operating equip.				$255,946,000

Table XVI-2. (Continued)

Acct. No.	Description	Quantity	Unit	Unit Price	Subtotal
30.0	Engineering and design				$ 15,425,000
31.0	Supervision and administration				$ 15,357,000
51.22	Operation & maintenance manual				$ 100,000
	TOTAL CONSTRUCTION				$286,828,000
	Lands				
	Damsite	125	Acre	$ 600.00	$ 75,000
	Admin. building area	1	Acre	$2,000.00	$ 2,000
	Borrow				
	Impervious	640	Acre	$4,000.00	2,560,000
	Pervious	325	Acre	$ 799.00	260,000
	Road easements	37	Acre	$ 297.30	11,000
	Flowage easements	61	Acre	$ 491.80	30,000
	Biological Mitigation				
	Endanagered Species (Eriastrum)	700	Acre	$3,500.000	2,450,000
	Others	847	Acre	L.S.	579,000
	Subtotal				$ 5,967,000
	Contingencies (20%)				1,193,000
	Relocations (PL91-646)				50,000
	Administrative				248,000
	Total, lands				$ 7,458,000
	Relocations (including contingencies)				
	So. California Edison				
	Powerhouse	1	Job	L.S.	$ 2,435,000
	Pressure Pipe	12,730	LF	$ 589.55	7,505,000
	Transmission line	3.5	Mile	$631,400.00	2,210,000
	Bear Valley Mutual water well	1	Job	L.S.	100,000
	San Bernardino Valley Water Conservation District diver. facilities	1	Job	L.S.	690,000
	Gravesites	1	Job	L.S.	50,000
	Total, relocations				$ 12,990,000
	Preconstruction engineering and design				$ 9,007,000
	TOTAL, FLOOD CONTROL FIRST COST				$316,283,000

Operation and Maintenance

16-04 Upon completion of the proposed flood control improvements, the annual operation and maintenance cost is estimated at $600,000. This estimated cost is comparable to the actual costs on similar types of improvements experienced by the Los Angeles District. The itemized cost for annual operation and maintenance is shown in table XVIII-1.

Comparison of Estimates

16-05 The first cost for the Seven Oaks Dam project estimated in the Supplement to Phase I GDM on the Santa Ana River Mainstem, including Santiago Creek, dated December 1985 (October 1985 Price Levels), is shown in table XVI-3. The present estimate and the Phase I GDM estimate at October 1987 price levels are shown for comparative purposes. The present estimate is $3,896,000 higher (table XVI-3) than the escalated Phase I GDM estimate.

Table XVI-3. Comparison of First Cost.

Acct. No.	Description	Phase I GDM Estimate (October 1985 Price Levels)	Phase I GDM Estimate (October 1987 Price Levels)	Present Estimate (October 1987 Price Levels)
03.	Reservoir clearing and grubbing	$67,000	$70,000	$67,000
04.	Dam	256,702,000	263,119,000	250,645,000
0.1	Main embankment[1]	(160,583,000)	(164,597,000)	(184,256,000)
0.2	Spillway	(70,678,000)	(72,445,000)	(35,133,000)
0.3	Outlet works	(25,441,000)	(26,077,000)	(31,256,000)
08.	Roads	0	0	3,956,000
18.0	Cultural resource preservation	0	0	200,000
19.0	Buildings, grounds & utilities	0	0	115,000
20.0	Permanent operating equipment	374,000	383,000	963,000
30.0	Engineering and design	17,720,000	18,295,000	15,425,000
31.0	Supervision and administration	12,660,000	13,071,000	15,357,000
51.22	Operation and maintenance manual	50,000	50,000	100,000
	Total, construction	287,573,000	294,988,000	286,828,000
	Lands[2]	4,803,000	4,924,000	7,458,000
	Relocations	11,800,000	12,095,000	12,990,000
	Preconstruction engineering & design	*	*	9,007,000
	Total, flood control first cost	$304,176,000	$312,007,000	$316,283,000
14.0	Recreation	371,000	380,000	0
	GRAND TOTAL PROJECT COST	$304,547,000	$312,387,000	$316,283,000

1. Includes Biological Mitigation Improvements.
2. Includes Biological Mitigation Lands.
*Included in Engineering and Design Costs.

16-06 The major differences between the escalated Phase I GDM estimate and the present estimate are explained as follows:

a. Dam. The decrease of $12,474,000 is the net effect of (1) $19,659,000 increase to the main embankment due to an increase in borrow material unit prices based on a more detailed estimate, (2) $37,312,000 decrease to the spillway due mainly to less costly excavation and handling costs based on geotechnical information regarding rock characteristics, and (3) $5,179,000 increase to the outlet works due to a more detailed estimate.

b. Roads. The increase of $3,956,000 is due to increased detail and an increase in quantities from the Phase I Supplemental GDM.

c. Cultural resource preservation. The increase of $200,000 is due to an added feature not included in the Phase I Supplement Report.

d. Buildings, grounds, and utilities. The increase of $115,000 is for an added feature not included in the Phase I Supplemental GDM.

e. Permanent operating equipment. The increase of $580,000 is due to an increase in geotechnical instrumentation and the addition of hydrologic facilities not included in the Phase I Supplemental GDM.

f. Engineering and design. The decrease of $2,870,000 is due to a decrease in rate applied to construction costs.

g. Supervision and administration. The increase of $2,286,000 is due to an increase in rate from 5 percent to 6 percent of the construction costs (including contingencies). This percentage is based on the actual prevailing rate experienced by the Los Angeles District Office.

h. Lands. The increase of $2,534,000 is due to a more detailed real estate appraisal.

i. Relocations. The increase of $895,000 is the net effect of (1) deletion of the U.S. Forest Service access road and replacement with the post project access roads in the Phase II design, (2) adverse sediment impacts to Southern California Edison facilities not previously considered, (3) more detailed design information on relocation alternatives, and (4) the relocation of grave sites not identified in the Phase I Supplemental GDM.

J. Preconstruction Engineering and Design. An increase of $9,007,000 results from separating out the preconstruction engineering and design costs from the engineering and design costs. This separation was not done for the Phase I estimate.

K. Recreation. The decrease of $380,000 is due to the deletion of recreation from the project.

XVII. DESIGN AND CONSTRUCTION SCHEDULE

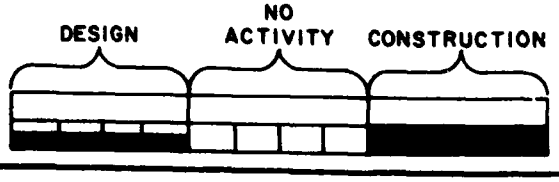
17-01 Preparation of Plans and Specifications. Preparation of contract plans and specifications for the construction of the proposed flood control project will be initiated after the Phase II General Design Memorandum for the Santa Ana River Mainstem, including Santiago Creek, is approved. Two contracts are scheduled prior to the outlet and main embankment construction. A test fill and an abutment excavation contract will be prepared from design information presented within this report.

17-02 Construction Schedule. Construction of the project will be scheduled to start in the spring of year 1. Initial construction will be the test fill and the abutment excavation. The main embankment and outlet will be scheduled for construction after an embankment and spillway Feature Design Memorandum and an outlet works Feature Design Memorandum are prepared and approved. The construction schedule shown in table XVII-1 may be modified as required based on total project requirements. The overall project construction schedule is provided in the main report.

17-03 Total Funds Required by Fiscal Years. Total funds including Federal and non-Federal share which will be required for the preparation of contract plans and specifications and for construction are shown in the main report. Table XVII-1 shows the total construction estimate and a generalized updated schedule.

LINE NO	UNIFORM COST CLASSIFICATION	FEATURE ITEMS	PROJECT COST ESTIMATE	TOTAL AS OF
1	03	RESERVOIR CLEARING	67	
2	04	DAM	250,645	
3	0.1	MAIN EMBANKMENT	(184,256)	
4	0.2	SPILLWAY	(35,133)	
5	0.3	OUTLET WORKS	(31,256)	
6	08	ROADS	3,956	
7	18	CULTURAL MITIGATION	200	
8	19	BUILDINGS, GROUNDS AND UTILITIES	115	
9	20	PERMANENT OPERATING EQUIPMENT	963	
10	30	ENGINEERING AND DESIGN	15,425	
11	31	SUPERVISION AND ADMINISTRATION	15,357	
12	51.22	OPERATION AND MAINTENANCE MANUAL	100	
13				
14		TOTAL CONSTRUCTION	286,828	
15				
16		LANDS	7,458	
17		RELOCATIONS	12,990	
18		PRE-CONSTRUCTION ENGINEERING AND DESIGN	9,007	
19				
20		TOTAL, FLOOD CONTROL FIRST COST	316,283	
21				
22				
23				
24				
25				
26				
27				

FUNDS IN THOUSANDS OF DOLLARS



SPL 1 FORM 571 NOV 72

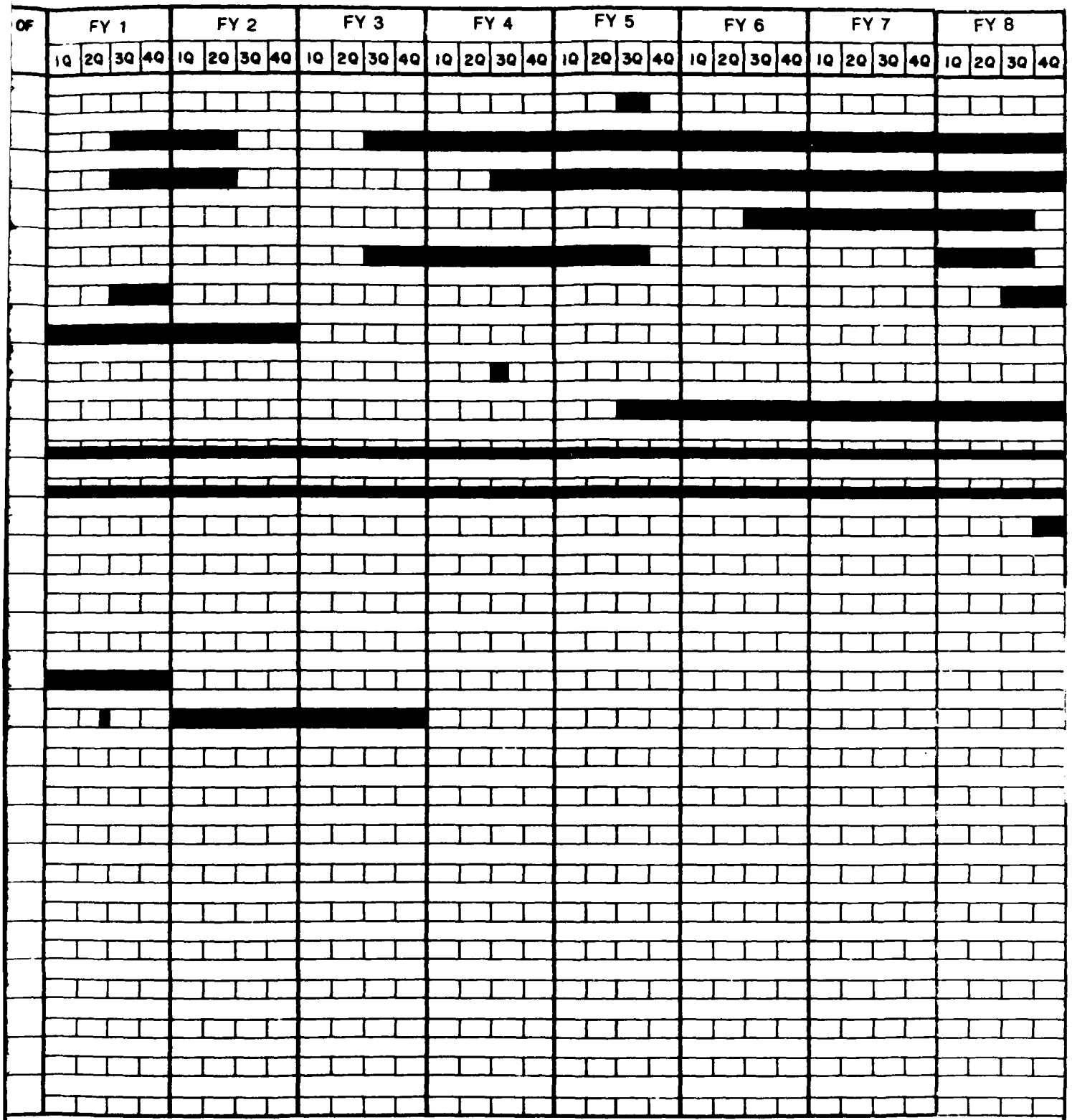


TABLE XVII-I

SEVEN OAKS DAM

DESIGN AND CONSTRUCTION SCHEDULE

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS

TO ACCOMPANY DESIGN MEMORANDUM NO.
DATED AUGUST 1988 SHEET 1 OF 1

XVIII. OPERATION AND MAINTENANCE

18-01 The dam will be operated as a single purpose flood control reservoir. Some incidental water conservation benefits will be derived from the operation of a debris pool from the end of each flood season to the month of October. The water conservation benefit will stem from the prolonged storage of floodwaters for release in the summer and early fall months when irrigation demands are highest. Detailed operation guidelines and criteria are presented in the Seven Oaks Dam Outlet Works design report appendix B of this volume, and the Water Control Plan in section XIX of this volume.

18-02 The Corps will develop operations and maintenance requirements and a decision will be made in the future as to who will actually perform the work.

18-03 The required operations and maintenance of all access roads will also be developed by the Corps and a decision will be made in the future as to who will actually perform the work. The United States Forest Service and Southern California Edison Company will continue to be allowed to use the access roads for the management of their lands and facilities upstream from the dam.

18-04 Development and execution of a Memorandum of Understanding between the Chief of Engineers and the Chief of the Forest Service will be undertaken which will identify management, financial, operation, and maintenance responsibilities, and identify criteria for future development of recreation in accordance with the Memorandum of Agreement, dated August 13, 1964, signed by the Secretaries of the Army and Agriculture.

Operation and Maintenance Manual

18-05 An operation and maintenance (O&M) manual would be prepared after construction of the flood control improvements in accordance with ER 1130-2-304 "Project Operations" and applicable provisions of ER 1150-2-301 "Local Cooperation." The estimated cost of the O&M manual is $100,000. Upon completion of the proposed flood control improvements, the annual operation and maintenance cost is estimated at $600,000.

This estimated cost is comparable to the actual costs on similar types of improvements experienced by the Los Angeles District. The local sponsors would be responsible for the operation and maintenance of the flood control improvements. The major items of operation and maintenance and their estimated annual costs are shown in table XVIII-1.

Operation of Outlet Gates

18-06 The water control plan for reservoir operation is presented in detail in section XIX of this volume. The plan requires that a dam tender be present at the dam for all gate opening changes. Remote gate control capability from the Reservoir Operation Center (ROC) in the Los Angeles District Office is not included in the plan. As with other dams operated by the Los Angeles District, gate opening changes will be transmitted by radio communication between the dam tender and the District's ROC.

Flood Emergency Plan

18-07 A flood emergency plan will be developed in accordance with HEC guidelines for Corps of Engineers' dams. The plan will be similar in format to the Emergency Action Plan During Construction, which is included as appendix C to this volume. The plans will cover emergency action response for earthquakes, spillway discharge without a dam breach, a dam breach with pool at spillway crest, Probable Maximum Flood (PMF) spillway discharge with a dam breach, and security alerts. The cost of the flood emergency plan is estimated to be $50,000. The inundation maps will be provided to local emergency service officials for evacuation planning and implementation. A set of briefing materials will also be developed to provide a better understanding of the basis for the flood studies and the flooding scenarios depicted in the inundation maps.

Water Quality Monitoring Program

18-08 A water quality monitoring program will be initiated at Seven Oaks Reservoir to establish a data base on those chemical, limnological, and bacteriological parameters that could adversely impact the environment in the Upper Santa Ana River Canyon. The parameters should be monitored during the months of January, April, May, June, and October. The results of the water quality monitoring program will be analyzed each year to determine necessary changes to the following year's monitoring program.

Table XVIII-1. Annual Operation and Maintenance Cost.
(October 1987 Price Levels)

Decription	Amount
Operation	
Condition surveys	$ 10,000
Periodic inspections	12,000
Dam tenders (2)	125,000
Communication equipment and hydrologic facilities	50,000
Monumented sediment ranges	3,000
Geotechnical instrumentations	31,000
Environmental Management	20,000
Water quality monitoring	20,000
Supervision and administration	
Reservoir regulation unit (District)	52,000
Contract supervision	24,000
Maintenance	
Dam embankment	20,000
Spillway	5,000
Outlet works	
Mechanical and electrical equipment	23,000
Debris removal and maintenance	25,000
Access road debris removal and maintenance	35,000
Major Replacement	
Gate repainting (15 years)	8,000
Outlet works tunnel invert overlay (25 years)	100,000
Access roads overlay (20 years)	37,000
Total, Annual Operation and Maintenance Cost	$600,000

XIX. RESERVOIR REGULATION

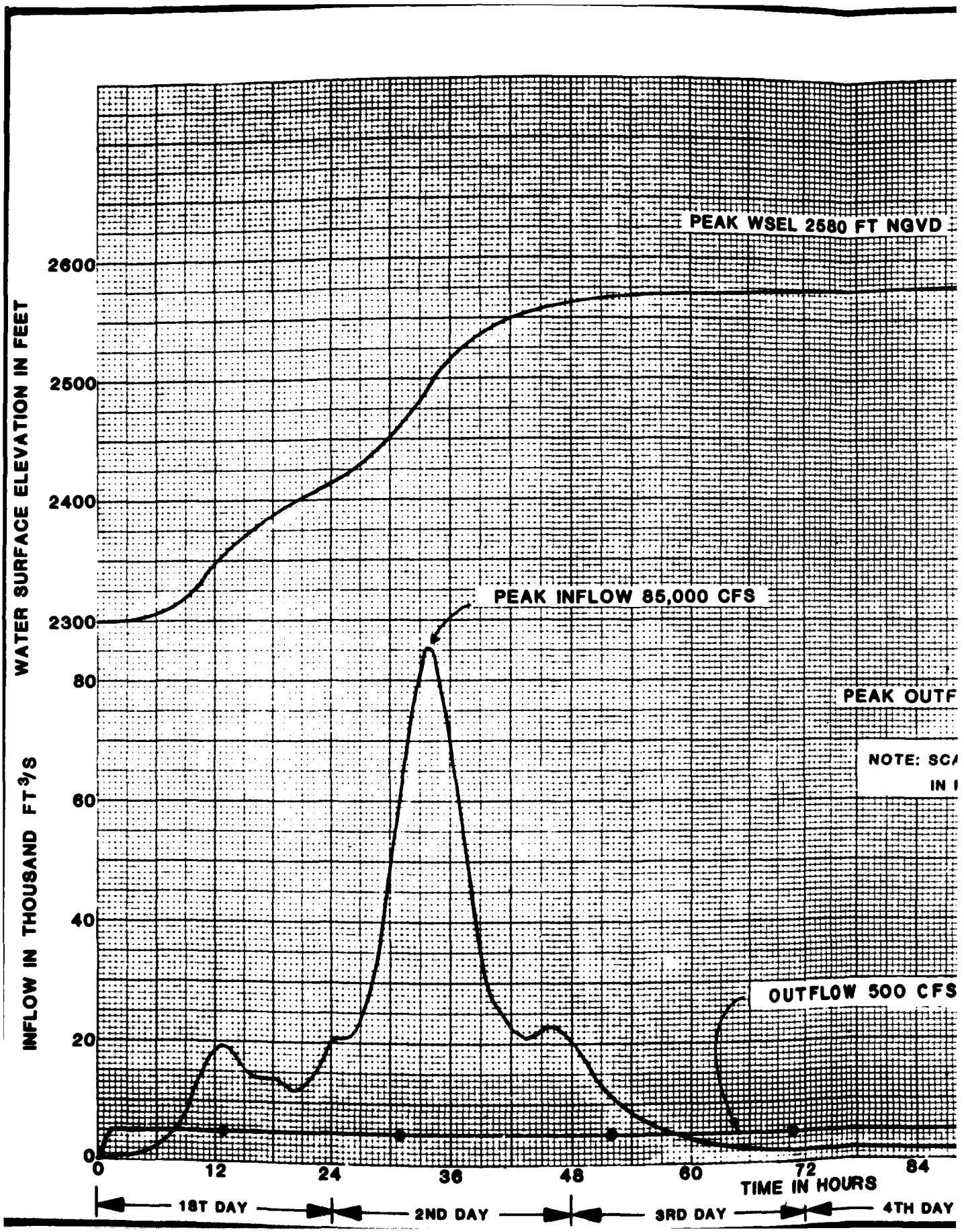
19-01 Storage allocations for the reservoir behind Seven Oaks Dam below spillway crest are as follows: flood control storage, 113,600 acre-feet and 100-year sediment storage, 32,000 acre-feet. The storage space allocated for sediment will be used for flood control until filled with sediment. Seven Oaks Dam flood control storage will be used to help control flooding on the lower Santa Ana River below Prado Dam by delaying inflow into Prado Reservoir, which will help control Prado's maximum water surface elevation. In addition, Seven Oaks Dam flood control storage will assist control of flooding on the Santa Ana River from the dam to Prado Dam. The conditions at Seven Oaks Dam, the local runoff from the drainage area between Seven Oaks Dam and Prado Dam, and the inflow, outflow and storage conditions at Prado Dam will all be used during an actual flood event to determine the proper release rate from Seven Oaks Dam. Stream and rainfall gauges in the Santa Ana River drainage area should be used when determining the proper release from Seven Oaks Dam.

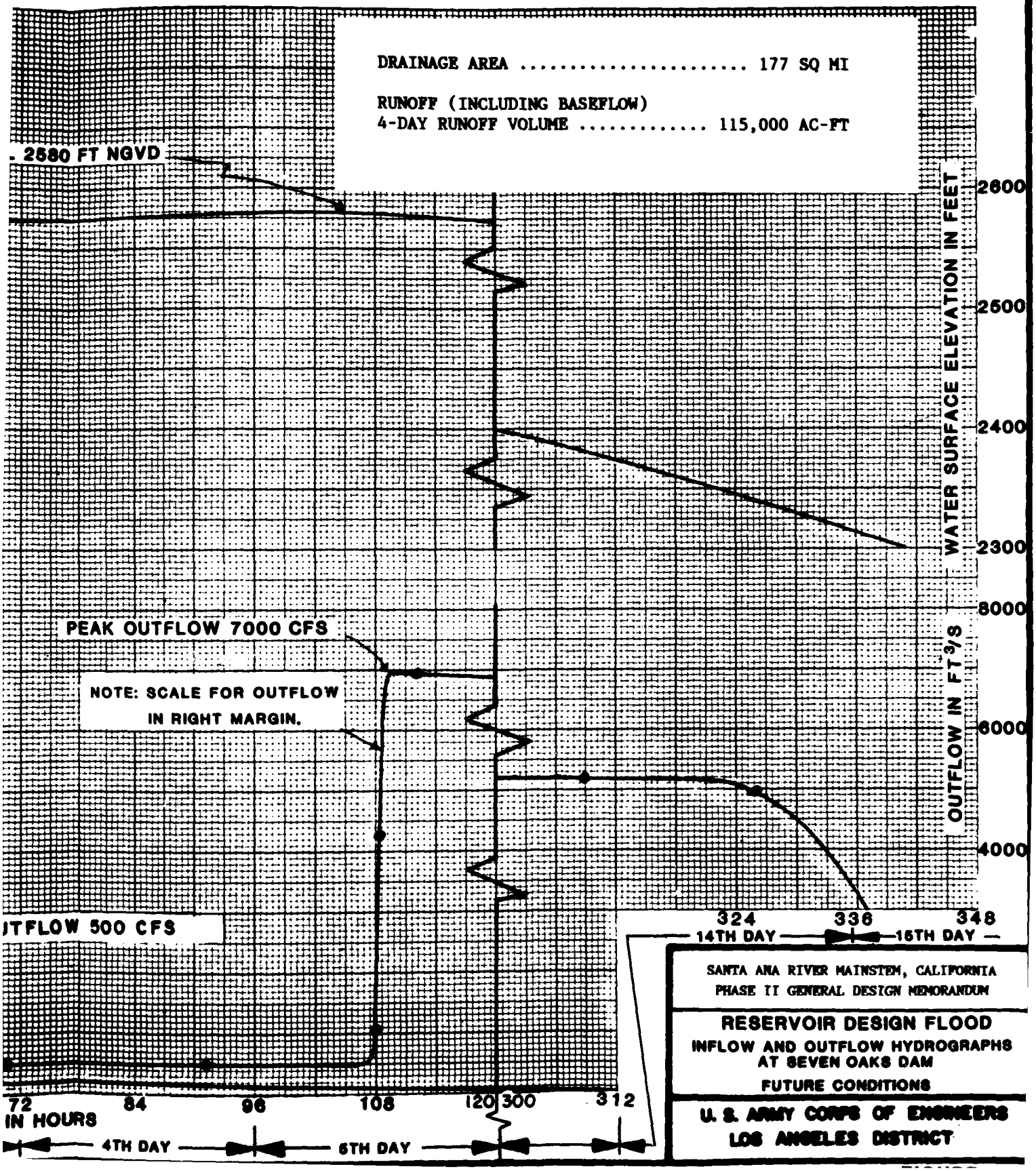
19-02 The debris pool elevation will be 2,200 feet NGVD during the first year after completion of the project construction. The debris pool elevation should be adjusted when needed during the project life based on the amount of accumulated sediment in the reservoir near the inlet structure. It is estimated that the elevation of the reservoir bottom near the inlet structure will raise about 165 feet due to sediment deposition by the end of project life. By then, it is estimated that the top of the debris pool would be elevation 2,300, the top of the trash rack.

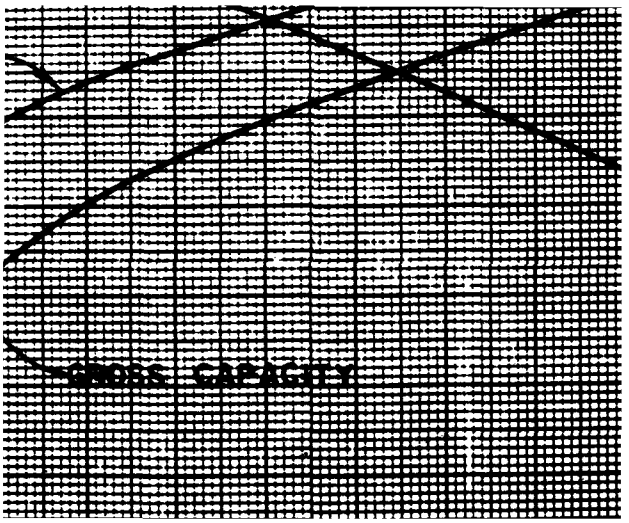
19-03 During the southern California rainfall-runoff season, any inflow to the reservoir will be stored for the purpose of forming the debris pool. There would not be any outflow until the top of the debris pool is reached. Once the runoff season has ended, releases will be made from the debris pool in cooperation with the existing groundwater recharge and water supply operations downstream of the dam. The reservoir should be dry by the end of August to allow for inspections and maintenance work.

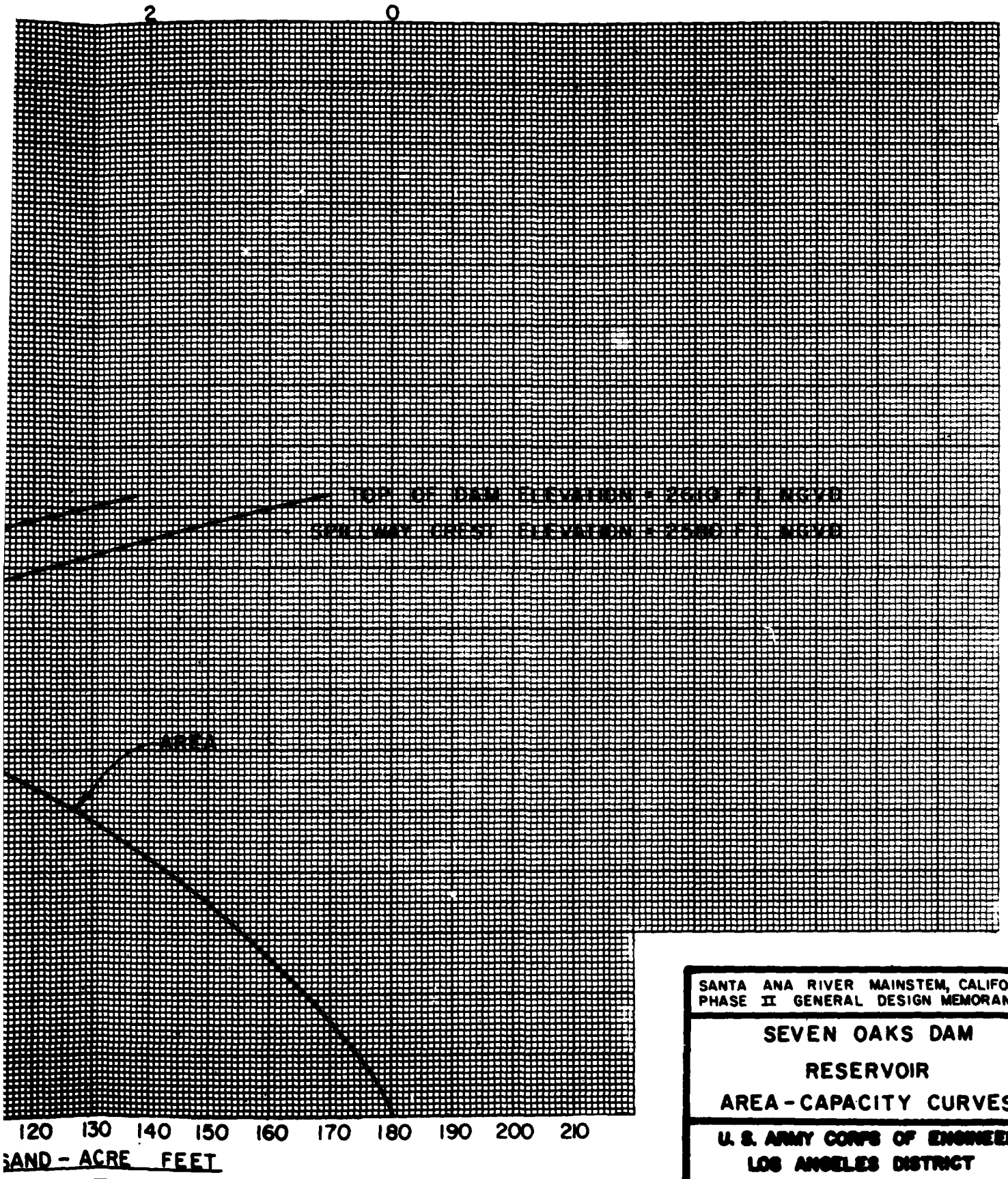
19-04 Once the debris pool elevation is reached, outflow from the reservoir will equal inflow up to a maximum of 500 ft^3/s until the water surface reaches elevation 2,265, which is the bottom of the trash rack. While the reservoir's water surface is on the trash rack, from elevation 2,265 to 2,299, the maximum release from Seven Oaks will be 50 ft^3/s. This lower release rate is necessary in order to keep floating material from collecting on the trash rack and blocking flow into the inlet structure. When the reservoir has risen above the trash rack at elevation 2,299, the release plan of outflow equals inflow up to a maximum of 500 ft^3/s will resume. This release will continue until it is determined that the flood threat at Prado Dam has passed. At that time, scheduled flow from Seven Oaks Dam will be increased in increments up to as much as 7,000 ft^3/s, to evacuate the reservoir in preparation for the next flood event. As the reservoir draws down to the trash rack, releases should again be reduced in order to prevent floating material from collecting on the rack.

19-05 The design plan of reservoir regulation described above was developed for this GDM. Actual regulation of Seven Oaks Reservoir will be in accordance with the water control plan and manual to be prepared by the Los Angeles District upon completion of the project construction. Water control plans developed by the District may vary somewhat from the "design" plan described above, but should not change the basic flood control criteria described. Additional information on Seven Oaks Dam reservoir regulation can be found in the Hydrology report, Volume 7 of this Phase II GDM.


PEAK WSEL 2580 FT NGVD
PEAK INFLOW 85,000 CFS
PEAK OUTF
NOTE: SCA
IN I
OUTFLOW 500 CFS
WATER SURFACE ELEVATION IN FEET
INFLOW IN THOUSAND FT3/S
2600
2500
2400
2300
80
60
40
20
0
0
12
24
36
48
60
72
84
TIME IN HOURS
1ST DAY
2ND DAY
3RD DAY
4TH DAY


DRAINAGE AREA ..... 177 SQ MI
RUNOFF (INCLUDING BASEFLOW)
4-DAY RUNOFF VOLUME ..... 115,000 AC-FT
. 2580 FT NGVD
PEAK OUTFLOW 7000 CFS
NOTE: SCALE FOR OUTFLOW IN RIGHT MARGIN.
ITFLOW 500 CFS
WATER SURFACE ELEVATION IN FEET
OUTFLOW IN FT3/S
2600
2500
2400
2300
8000
6000
4000
72
84
96
108
120
300
312
324
336
348
IN HOURS
4TH DAY
5TH DAY
14TH DAY
15TH DAY
SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM
RESERVOIR DESIGN FLOOD
INFLOW AND OUTFLOW HYDROGRAPHS
AT SEVEN OAKS DAM
FUTURE CONDITIONS
U. S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

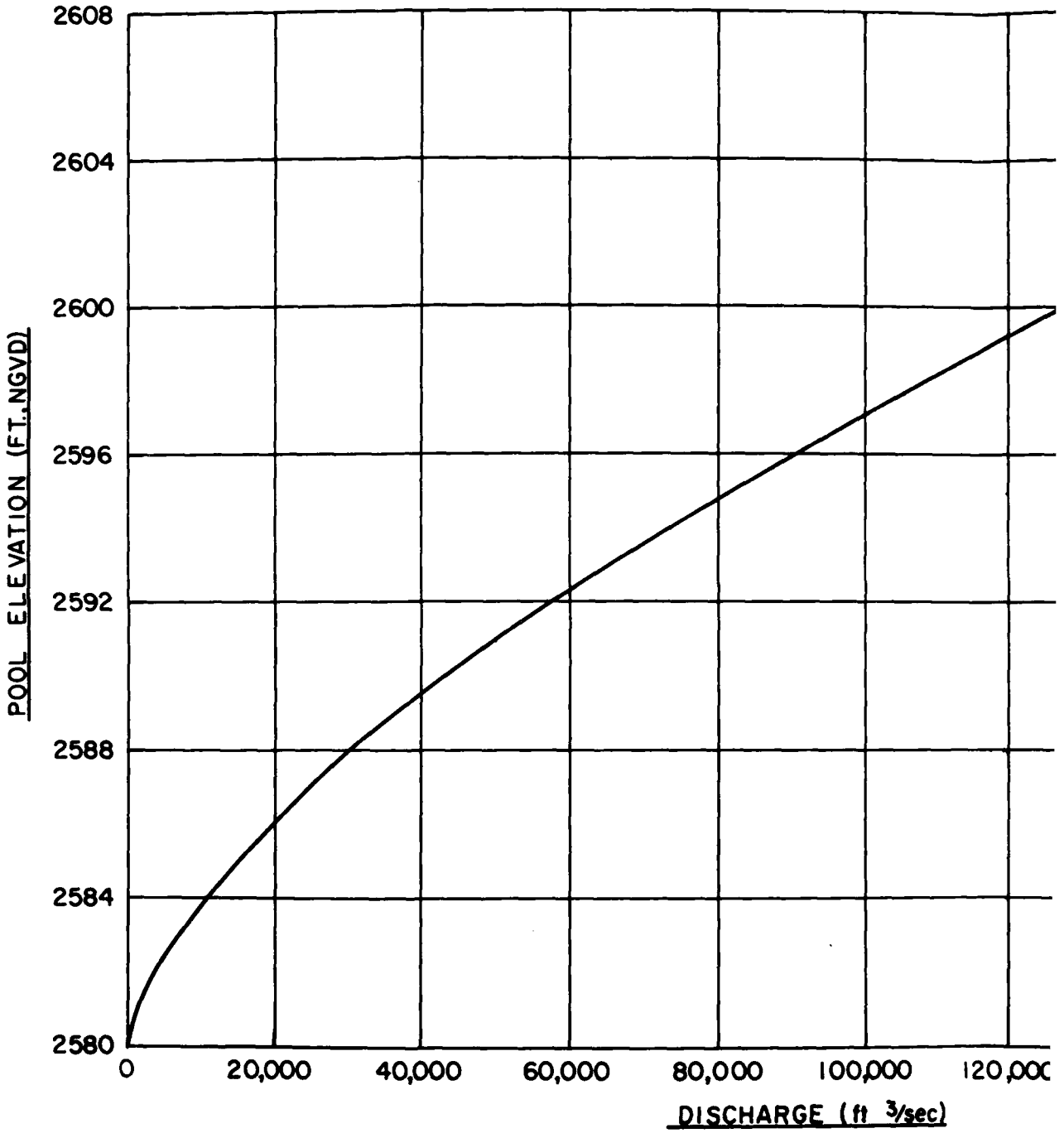

GROSS CAPACITY

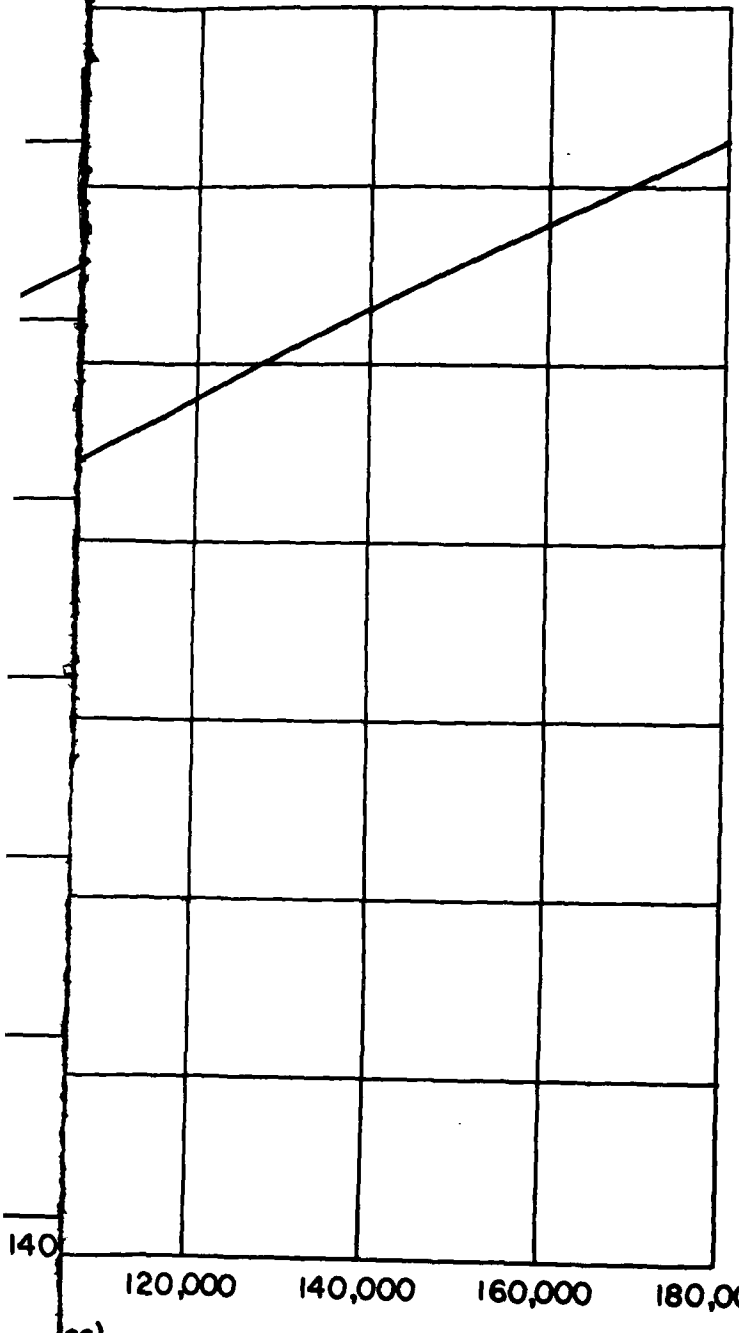


SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM
RESERVOIR
AREA-CAPACITY CURVES

U. S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT


2608
2604
2600
2596
2592
2588
2584
2580
POOL ELEVATION (FT., NGVD)
0
20,000
40,000
60,000
80,000
100,000
120,000
DISCHARGE (ft 3/sec)



SANTA ANA RIVER MAINSTEM CALIFORNIA,
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM
SPILLWAY RATING CURVE

US ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

FIGURE 3

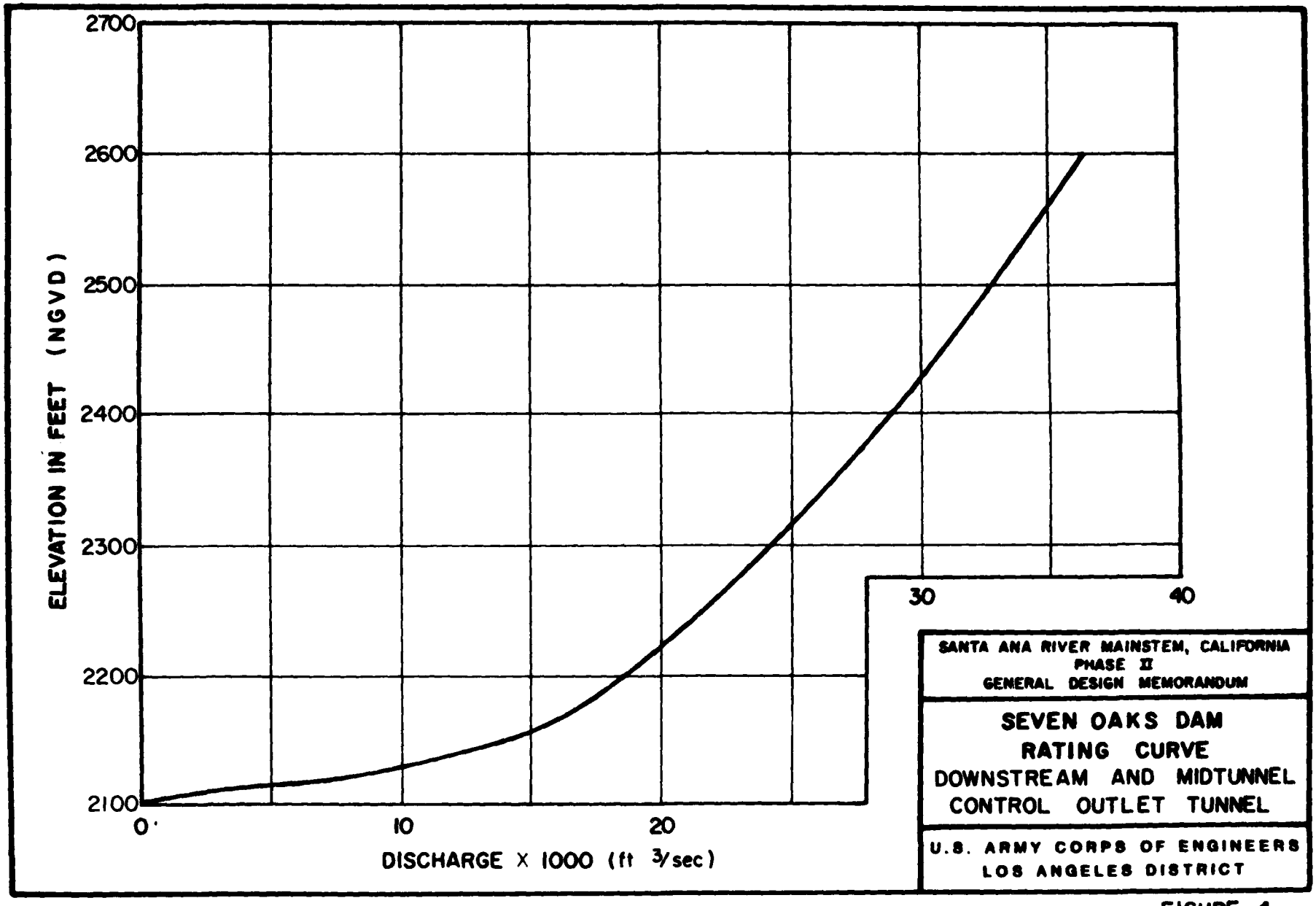

2700
2600
2500
2400
2300
2200
2100
ELEVATION IN FEET (NGVD)
0
10
20
30
40
DISCHARGE X 1000 (ft 3/sec)
SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II
GENERAL DESIGN MEMORANDUM
SEVEN OAKS DAM
RATING CURVE
DOWNSTREAM AND MIDTUNNEL
CONTROL OUTLET TUNNEL
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

FIGURE 4

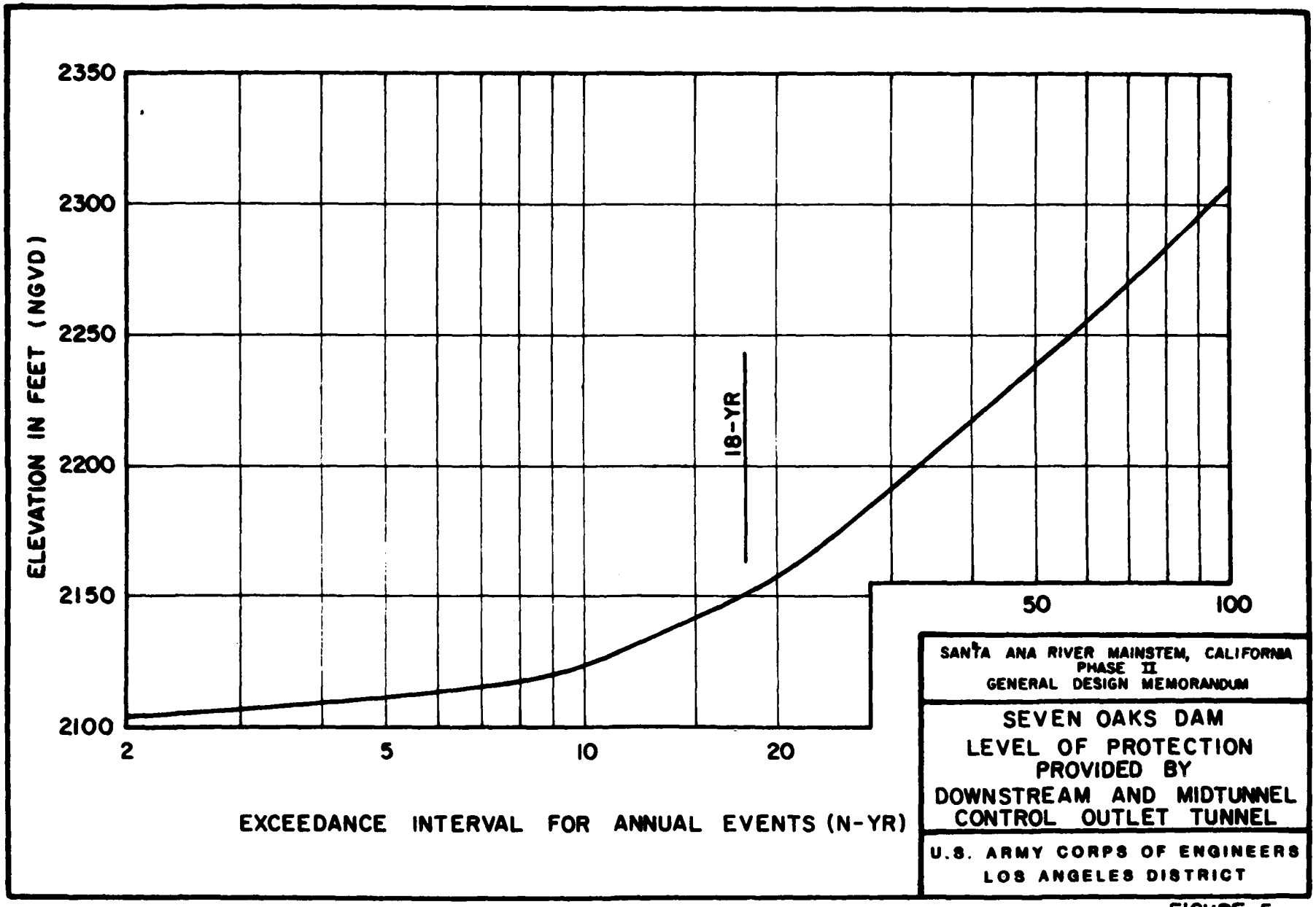

2350
2300
2250
2200
2150
2100
ELEVATION IN FEET (NGVD)
18-YR
2
5
10
20
50
100
EXCEEDANCE INTERVAL FOR ANNUAL EVENTS (N-YR)
SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II
GENERAL DESIGN MEMORANDUM
SEVEN OAKS DAM
LEVEL OF PROTECTION
PROVIDED BY
DOWNSTREAM AND MIDTUNNEL
CONTROL OUTLET TUNNEL
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

FIGURE 5

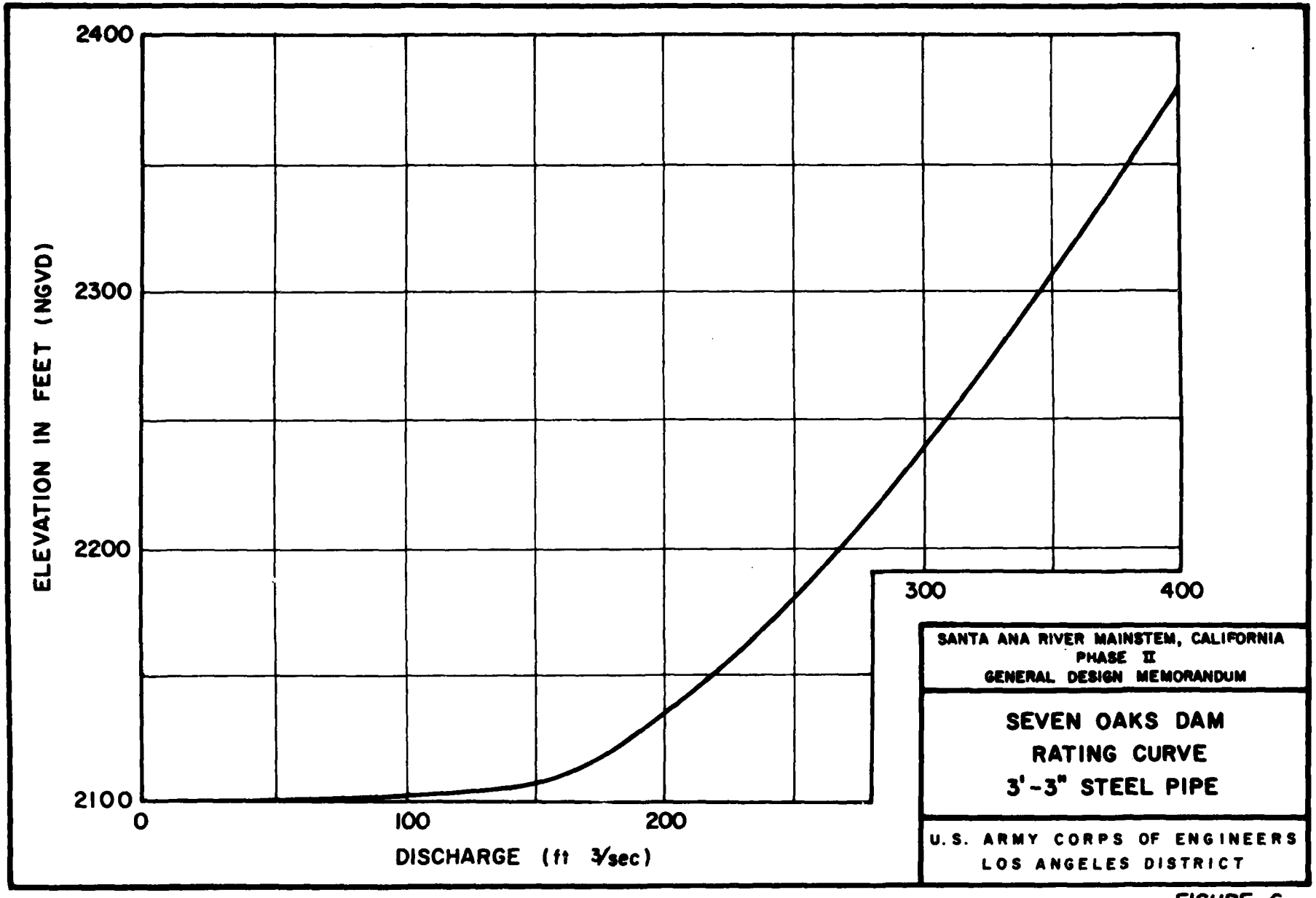

2400
2300
2200
2100
ELEVATION IN FEET (NGVD)
0
100
200
300
400
DISCHARGE (ft 3/sec)
SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II
GENERAL DESIGN MEMORANDUM
SEVEN OAKS DAM
RATING CURVE
3'-3" STEEL PIPE
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

FIGURE 6

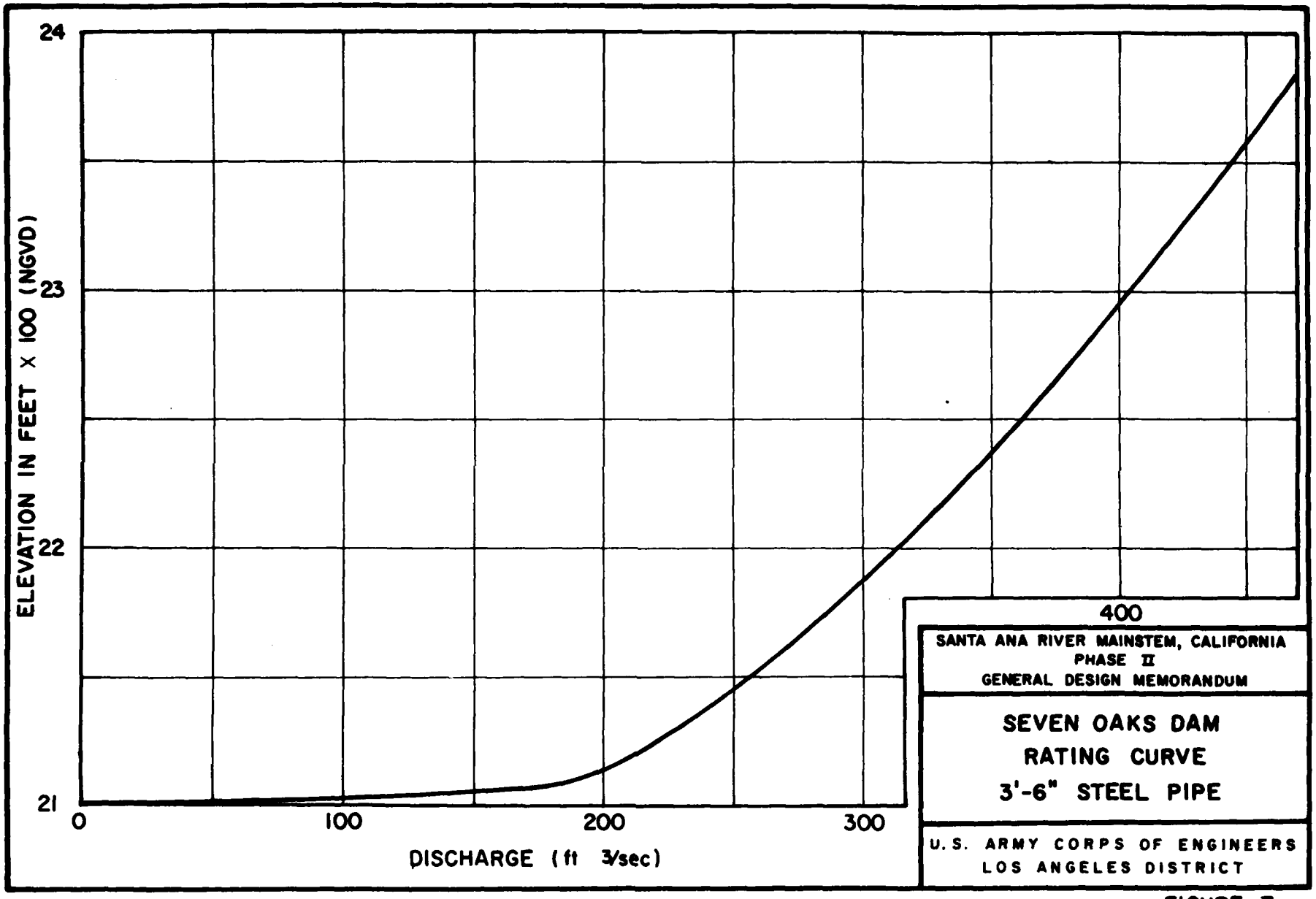

24
23
22
21
ELEVATION IN FEET X 100 (NGVD)
0
100
200
300
400
DISCHARGE (ft 3/sec)
SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II
GENERAL DESIGN MEMORANDUM
SEVEN OAKS DAM
RATING CURVE
3'-6" STEEL PIPE
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

FIGURE 7

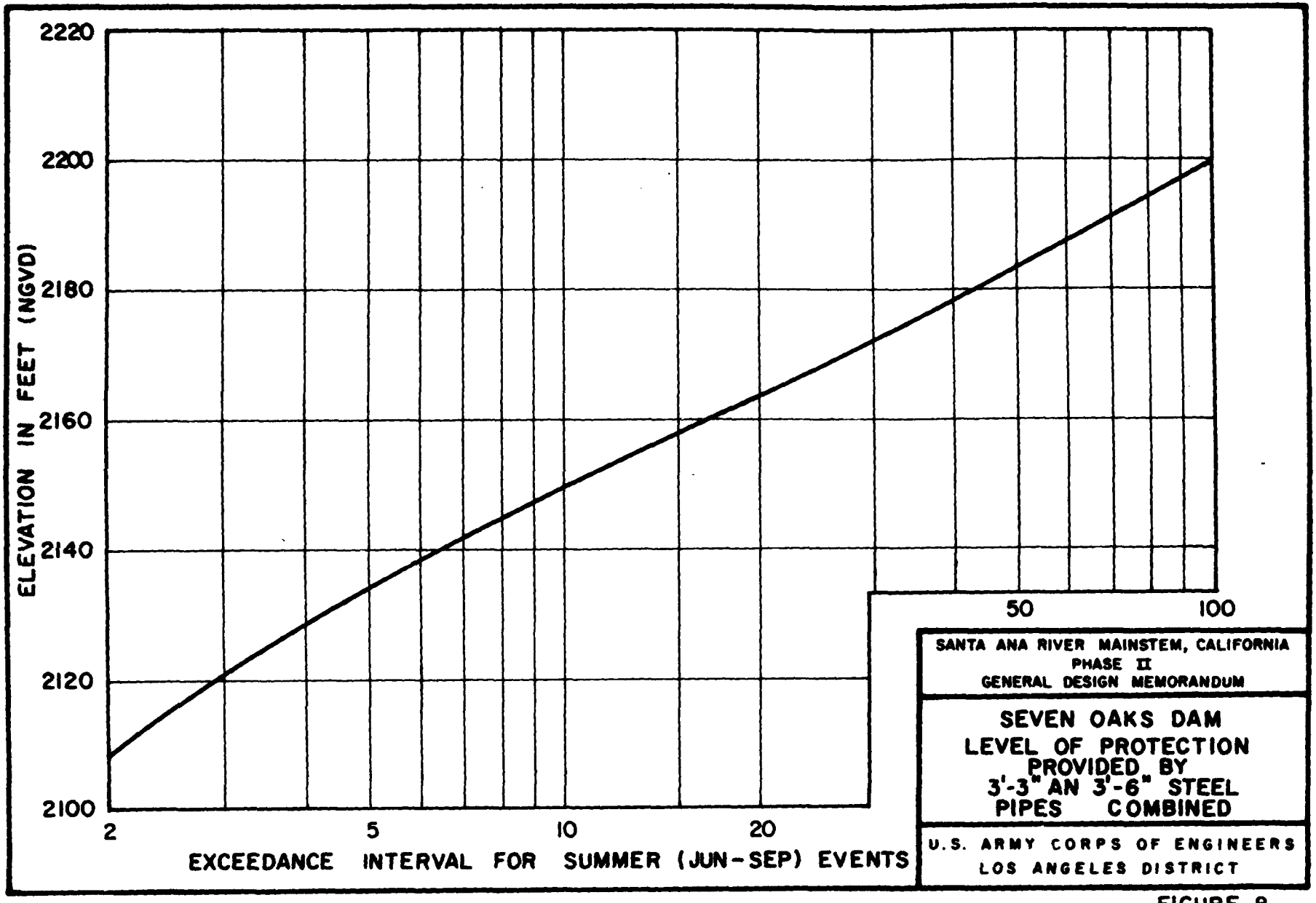

2220
2200
2180
2160
2140
2120
2100
ELEVATION IN FEET (NGVD)
2
5
10
20
50
100
EXCEEDANCE INTERVAL FOR SUMMER (JUN-SEP) EVENTS
SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II
GENERAL DESIGN MEMORANDUM
SEVEN OAKS DAM
LEVEL OF PROTECTION
PROVIDED BY
3'-3" AN 3'-6" STEEL
PIPES COMBINED
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

FIGURE 8

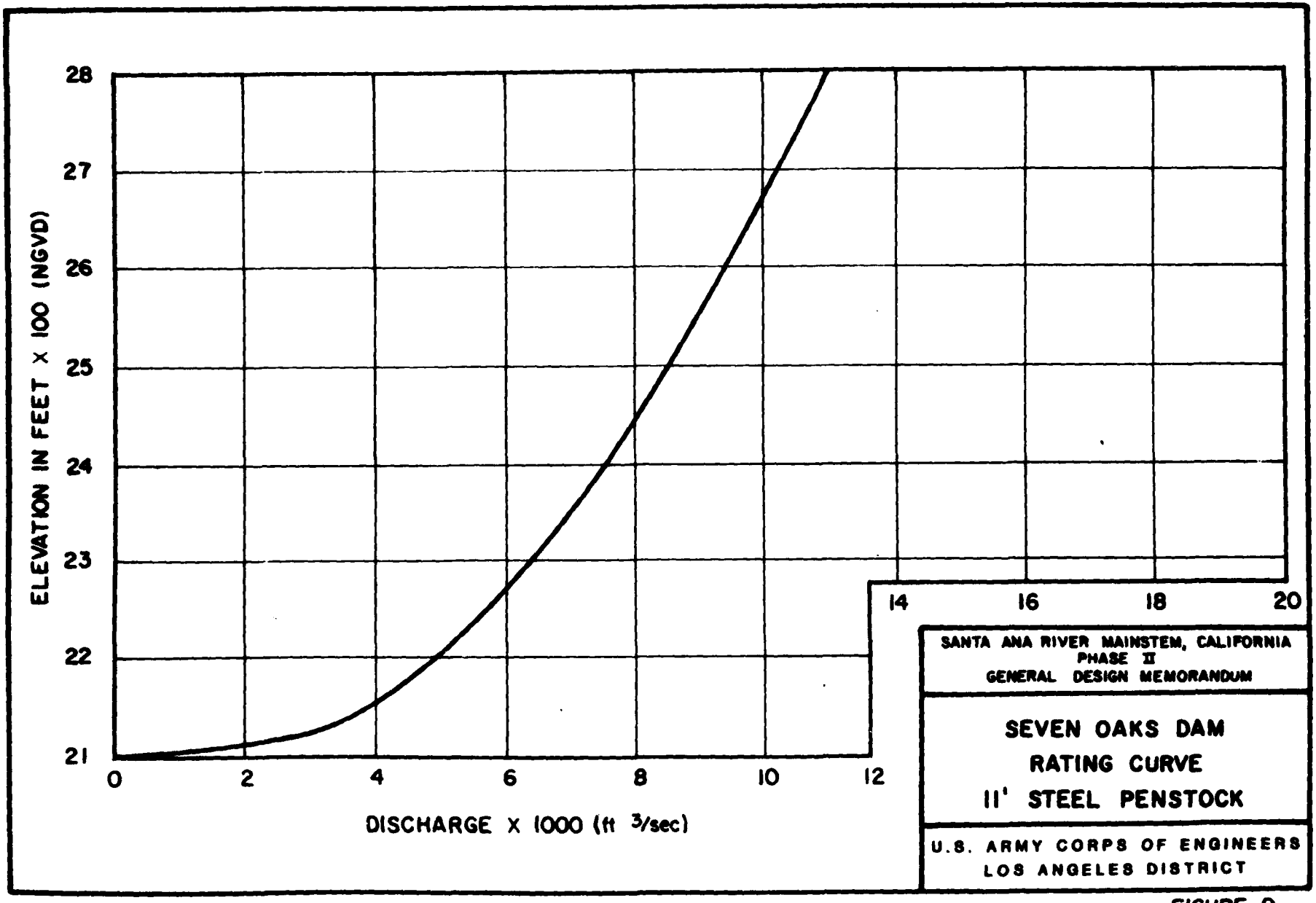

ELEVATION IN FEET X 100 (NGVD)
28
27
26
25
24
23
22
21
0
2
4
6
8
10
12
14
16
18
20
DISCHARGE X 1000 (ft 3/sec)
SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II
GENERAL DESIGN MEMORANDUM
SEVEN OAKS DAM
RATING CURVE
11' STEEL PENSTOCK
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

FIGURE 9

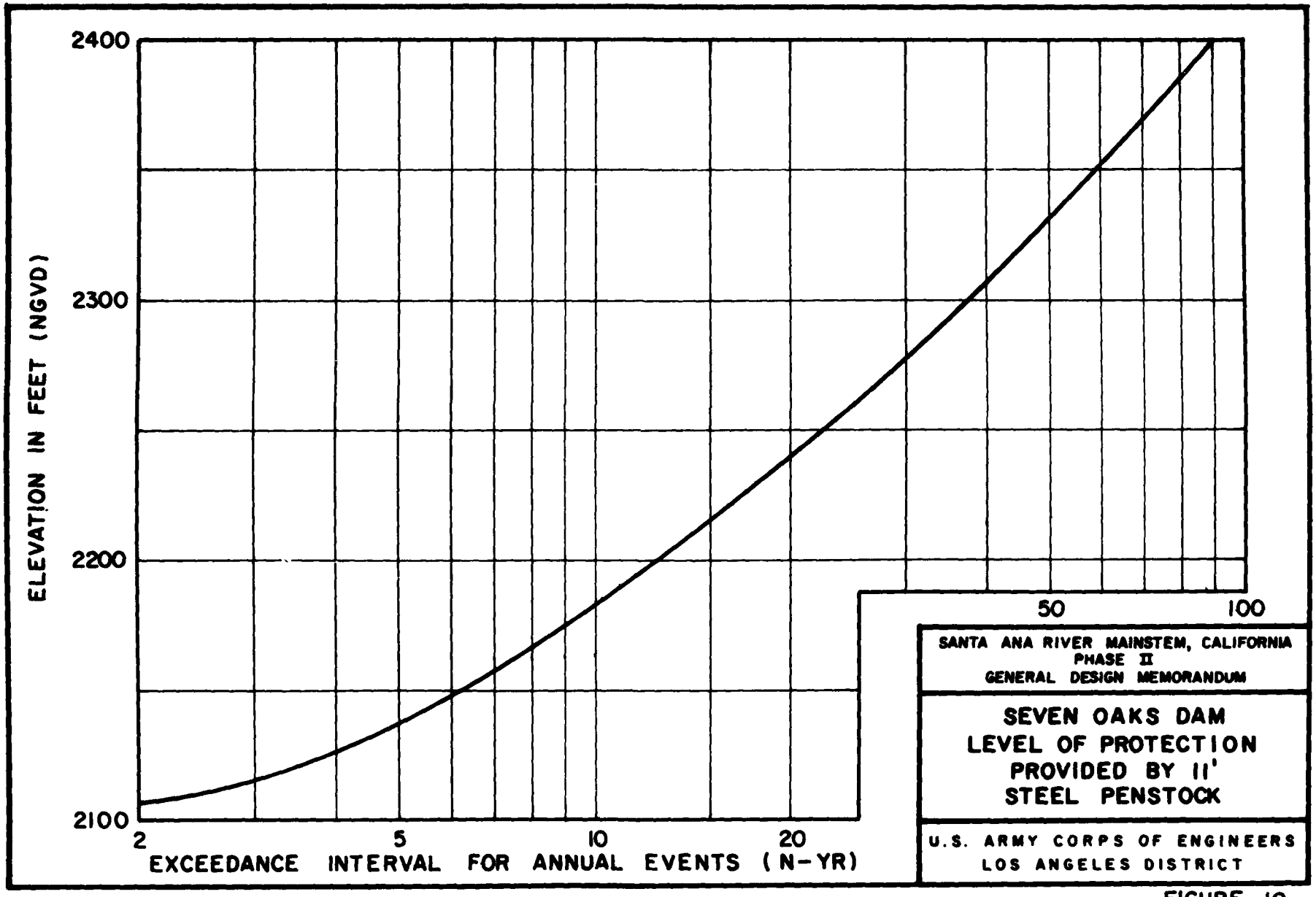

2400
2300
2200
2100
ELEVATION IN FEET (NGVD)
2
5
10
20
50
100
EXCEEDANCE INTERVAL FOR ANNUAL EVENTS (N-YR)
SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II
GENERAL DESIGN MEMORANDUM
SEVEN OAKS DAM
LEVEL OF PROTECTION
PROVIDED BY 11'
STEEL PENSTOCK
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

FIGURE 10

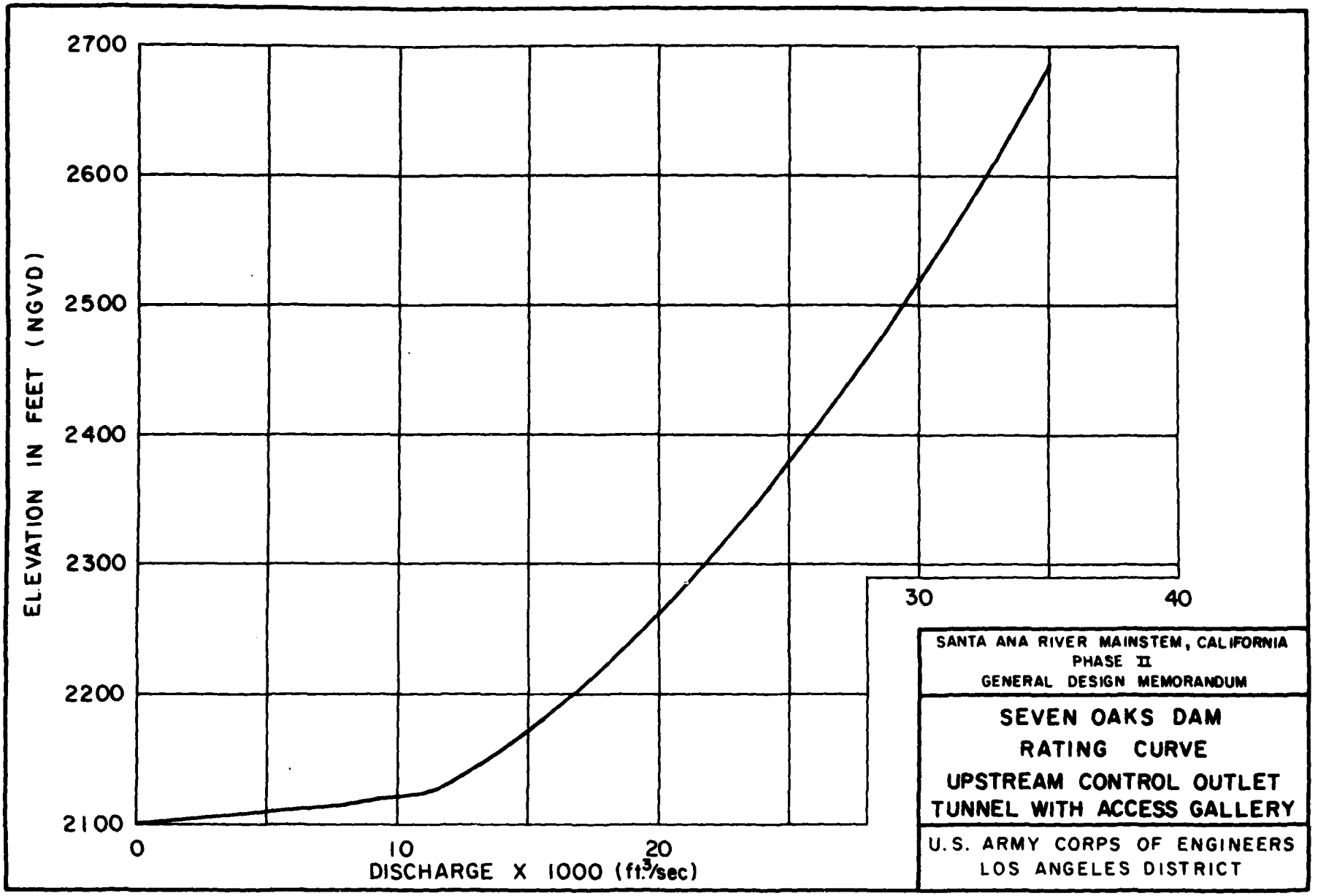

2700
2600
2500
2400
2300
2200
2100
ELEVATION IN FEET (NGVD)
0
10
20
30
40
DISCHARGE X 1000 (ft.³/sec)
SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II
GENERAL DESIGN MEMORANDUM
SEVEN OAKS DAM
RATING CURVE
UPSTREAM CONTROL OUTLET
TUNNEL WITH ACCESS GALLERY
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

FIGURE 11

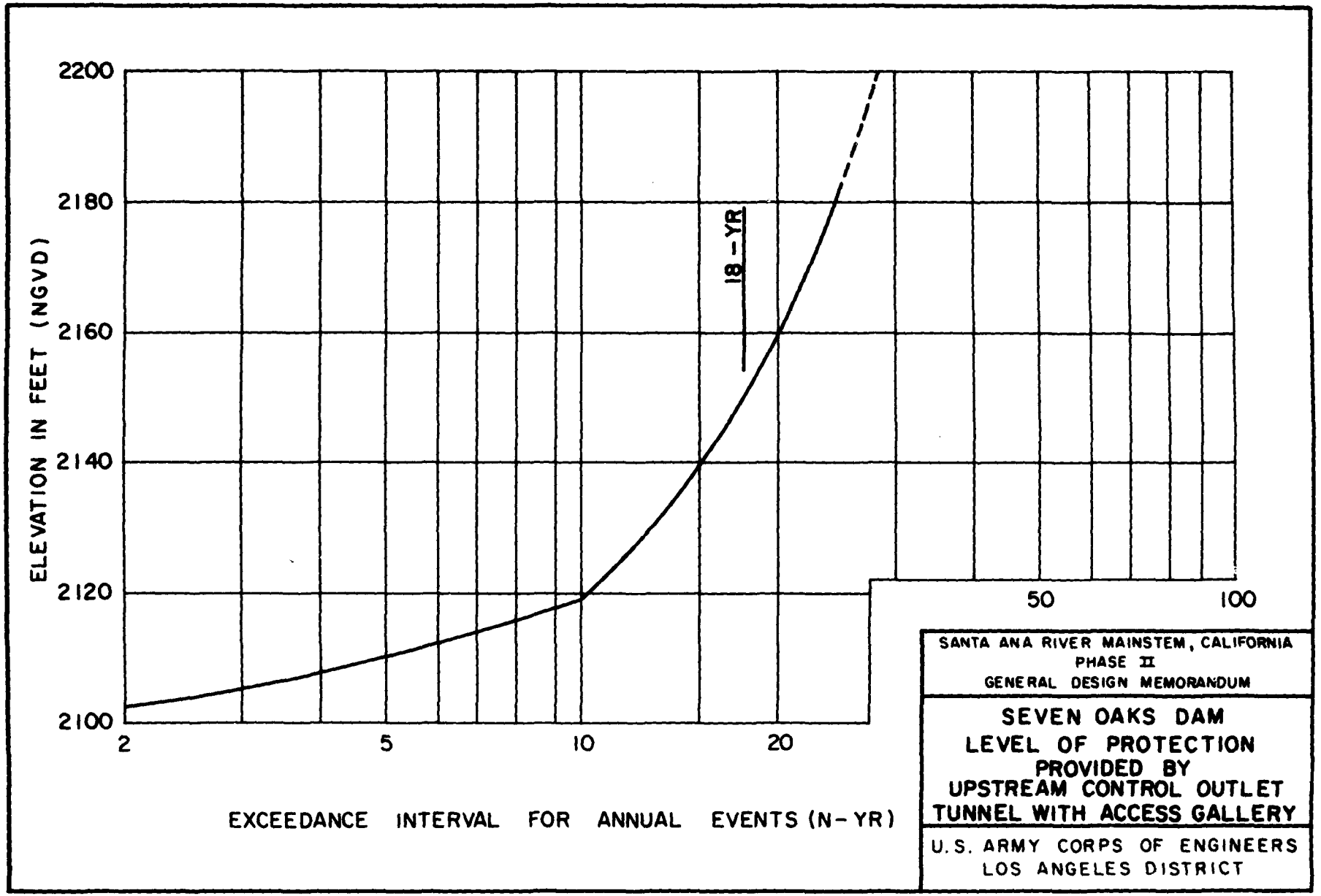
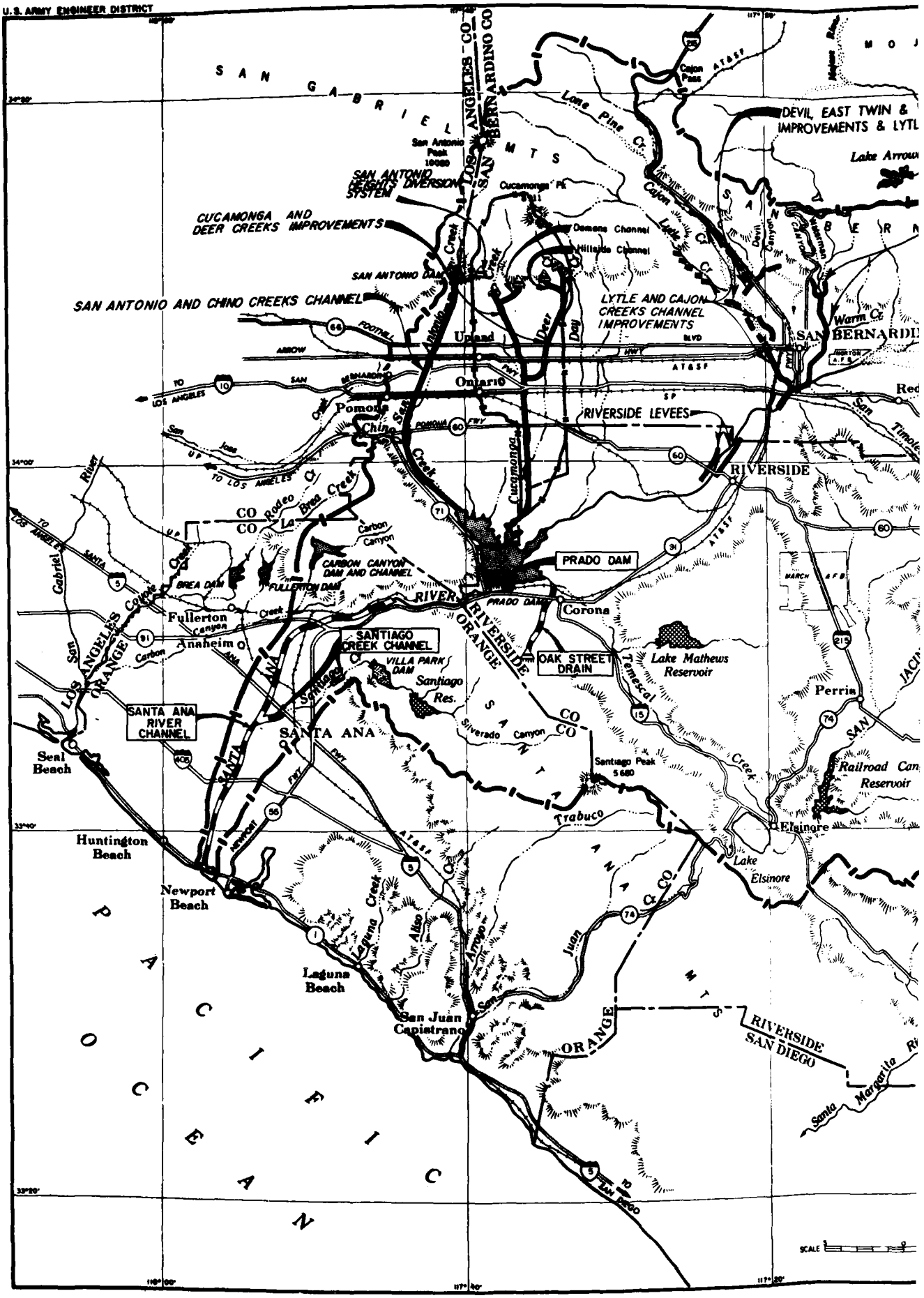
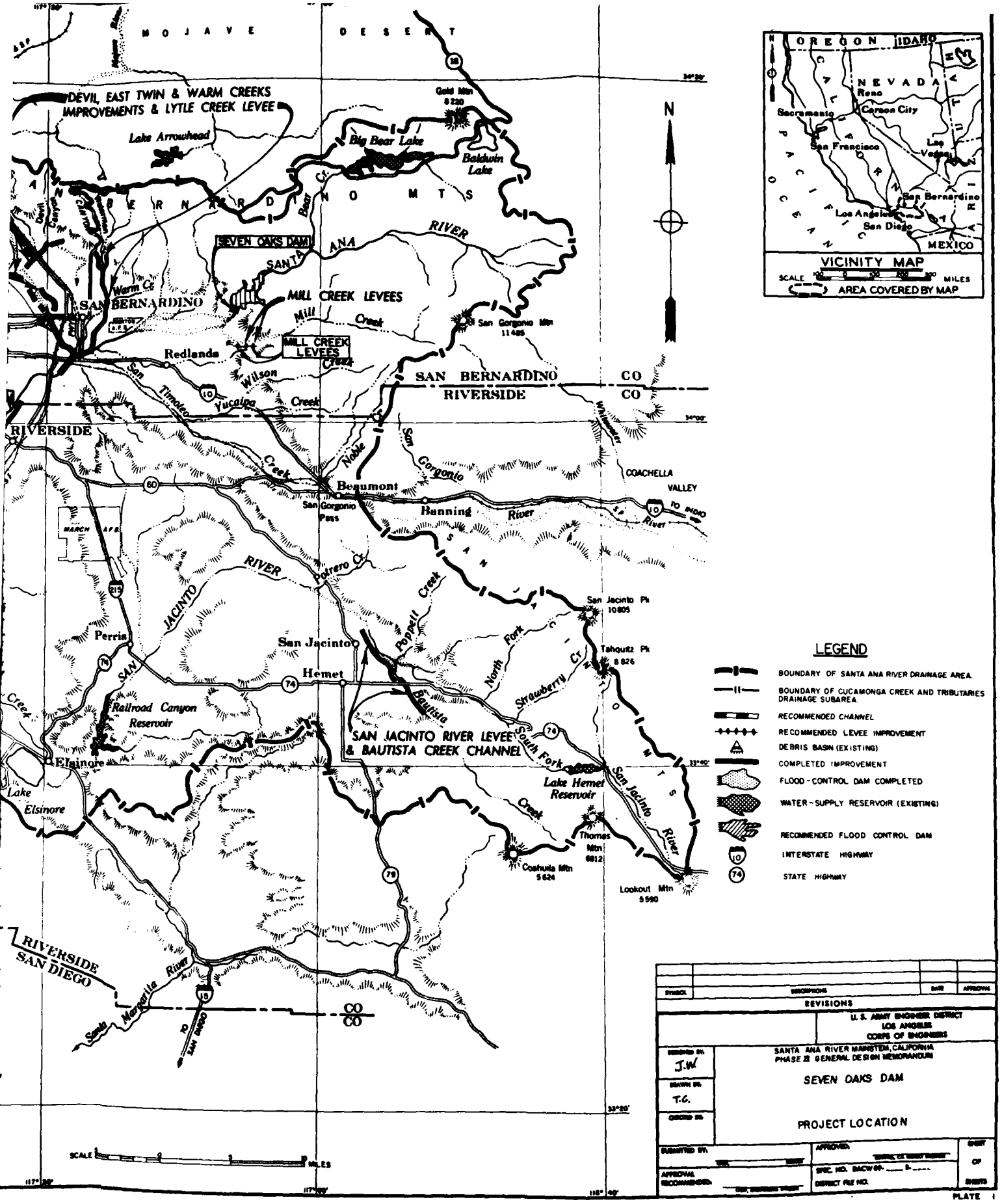

2200
2180
2160
2140
2120
2100
ELEVATION IN FEET (NGVD)
18-YR
2
5
10
20
50
100
EXCEEDANCE INTERVAL FOR ANNUAL EVENTS (N-YR)
SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II
GENERAL DESIGN MEMORANDUM
SEVEN OAKS DAM
LEVEL OF PROTECTION
PROVIDED BY
UPSTREAM CONTROL OUTLET
TUNNEL WITH ACCESS GALLERY
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

FIGURE 12


U.S. ARMY ENGINEER DISTRICT
SAN GABRIEL MTS
LOS ANGELES CO
SAN BERNARDINO CO
San Antonio Peak 10080
SAN ANTONIO HEIGHTS DIVERSION SYSTEM
CUCAMONGA AND DEER CREEKS IMPROVEMENTS
SAN ANTONIO AND CHINO CREEKS CHANNEL
SAN ANTONIO DAM
Cucamonga Pk.
Cajon Pass
Lone Pine Cr.
Lytle Cr.
Cajon Cr.
Devil Canyon
Waterman Canyon
Domenq Channel
Hillside Channel
DEVIL, EAST TWIN & IMPROVEMENTS & LYTL
Lake Arrowh
LYTLE AND CAJON CREEKS CHANNEL IMPROVEMENTS
Warm Cr.
SAN BERNARDI
Upland
Ontario
Pomona
Chino
RIVERSIDE LEVEES
RIVERSIDE
Red
San Timoteo
PRADO DAM
CARBON CANYON DAM AND CHANNEL
BREA DAM
FULLERTON DAM
Fullerton
Anaheim
Corona
SANTIAGO CREEK CHANNEL
VILLA PARK DAM
OAK STREET DRAIN
Lake Mathews Reservoir
Temescal
Perris
SANTA ANA RIVER CHANNEL
SANTA ANA
Santiago Res.
Silverado Canyon
Santiago Peak 5 680
Railroad Can
Reservoir
Seal Beach
Huntington Beach
Newport Beach
Laguna Beach
Trabuco
Elsinore
Lake Elsinore
San Juan Capistrano
ORANGE
RIVERSIDE
SAN DIEGO
Santa Margarita
PACIFIC OCEAN
SCALE



SYMBOL	DESCRIPTION	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY: J.W
DRAWN BY: T.G.
CHECKED BY:

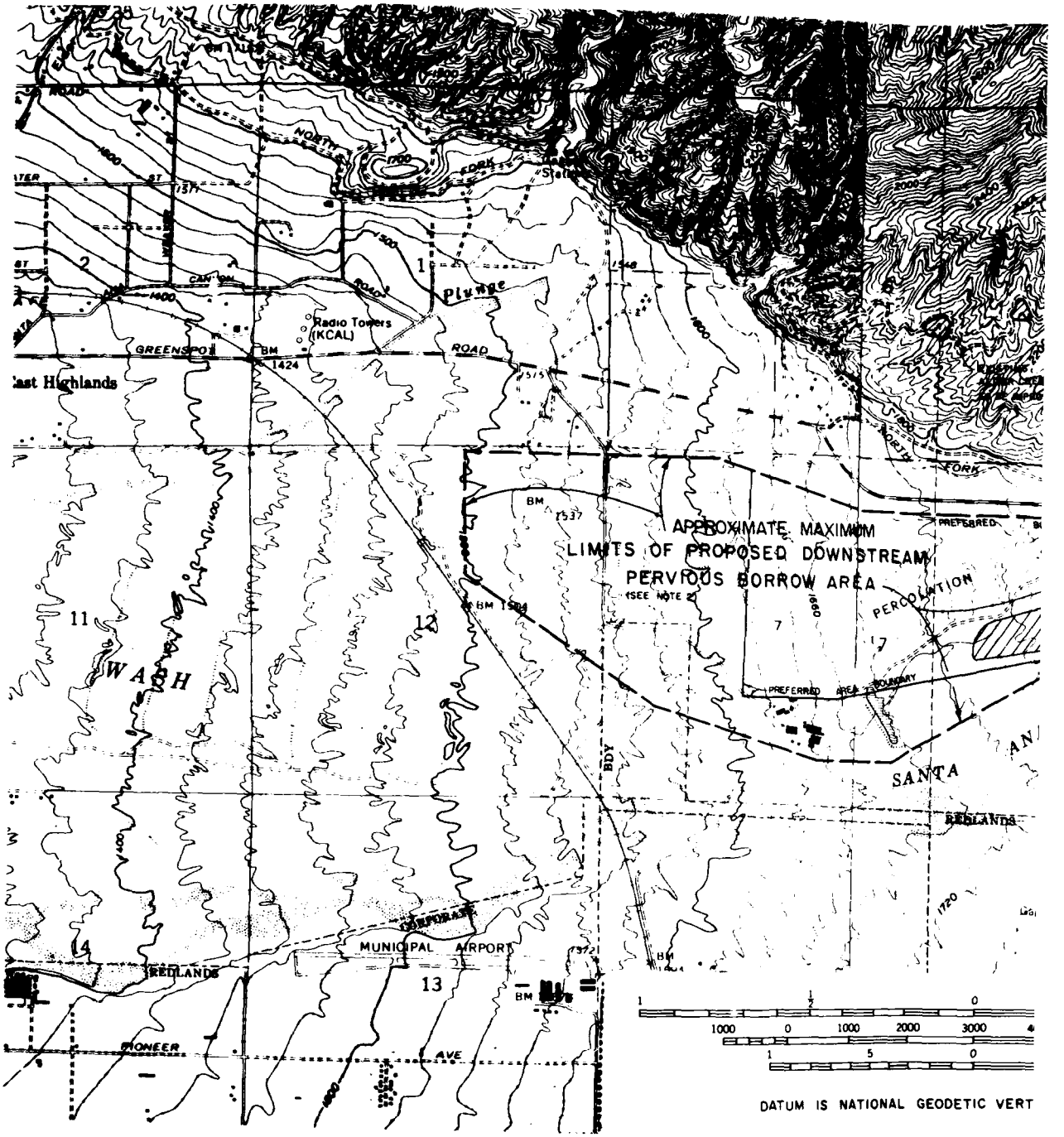
SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

PROJECT LOCATION

SUBMITTED BY: DATE
APPROVED: DIVISION, CE, ENGINEER
APPROVAL RECOMMENDED: CHIEF, ENGINEERING DIVISION
SPEC. NO. DACW09-__-__-__
DISTRICT FILE NO.
SHEET OF SHEETS

PLATE 1



DATUM IS NATIONAL GEODETIC VERT

N

NOTES:

1. EXISTING BEAR VALLEY HIGH LINE AQUEDUCT AND THE GREENSPOT PIPELINE TO BE PROTECTED IN PLACE OR AVOIDED INSIDE THE IMPERVIOUS BORROW AREA. ALSO SEE PLATES 25, 26 AND GEOTECHNICAL APPENDIX A, PLATE A-60 FOR FURTHER DETAILS.
2. EXISTING WATER PERCOLATION BASINS AND DIVERSION FACILITIES TO BE REPLACED. SEE GEOTECHNICAL APPENDIX A, PLATE A-101, FOR DETAILS.

SOURCE OF INFORMATION

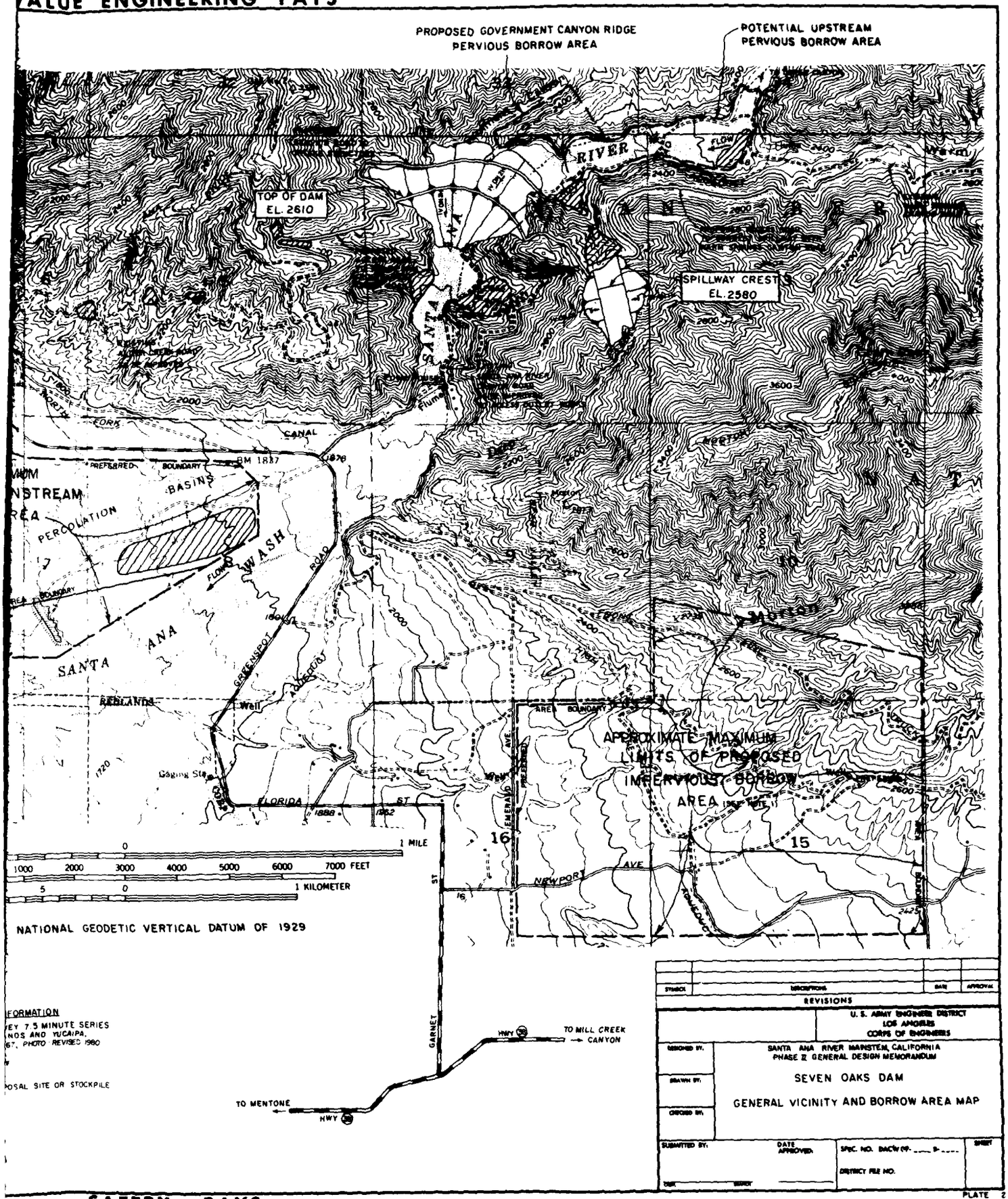
U.S. GEOLOGICAL SURVEY 7.5 MINUTE SERIES QUADRANGLES OF REDLANDS AND YUCAIPA, CALIFORNIA, DATED 1967, PHOTO-REVISED 1980

LEGEND

PROPOSED DISPOSAL SITE OR STOCKPILE AREA

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

VALUE ENGINEERING PAYS



SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

GENERAL VICINITY AND BORROW AREA MAP

SUBMITTED BY: DATE APPROVED:

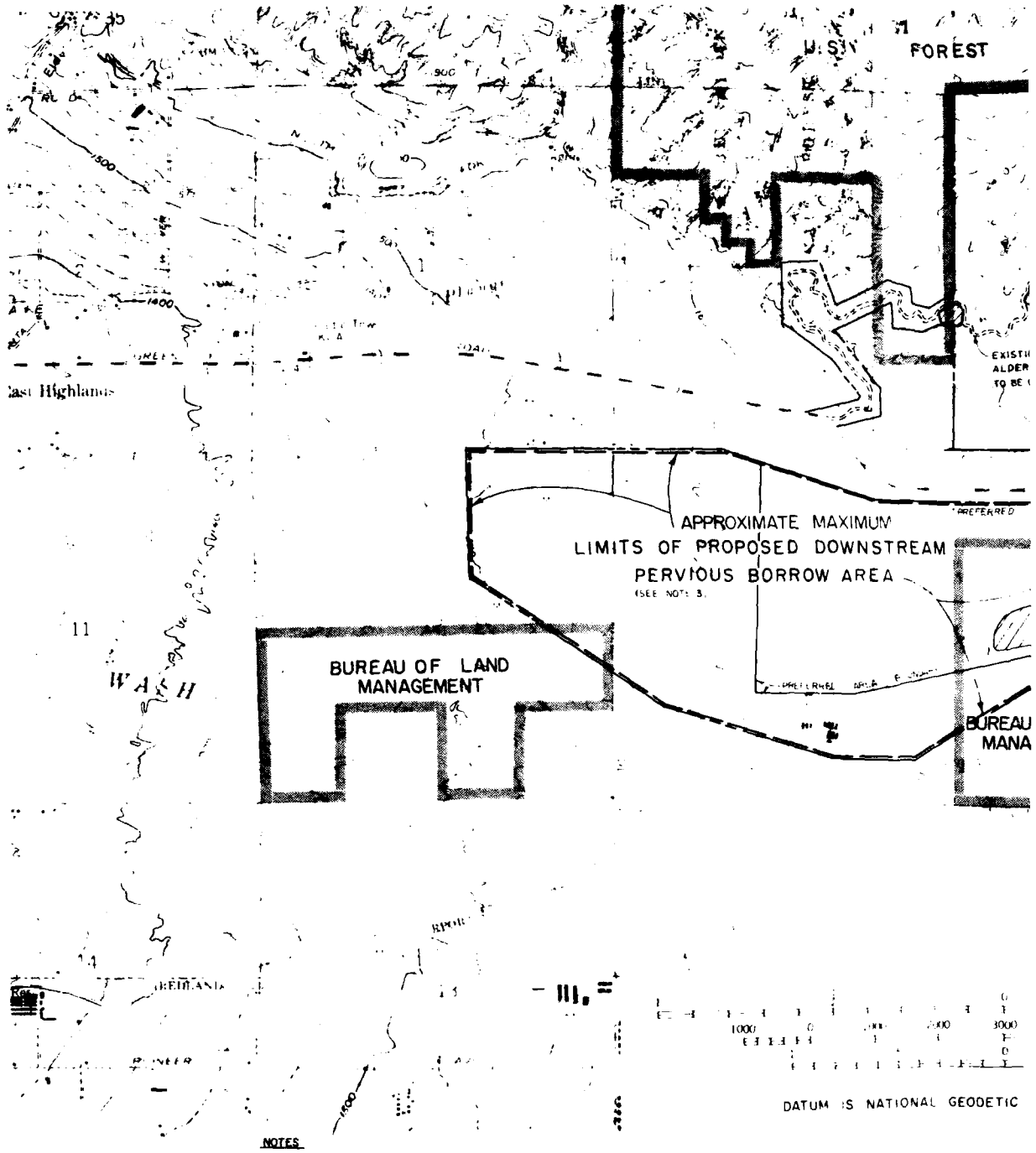
SPEC. NO. DACW09-__-__-____

DISTRICT FILE NO.

SHEET

PLATE 2

SAFETY PAYS



ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

NOTES

1-THE PROPOSED RIGHTS-OF-WAY SHOWN ARE BASED ON AVAILABLE DESIGN DATA AND ARE SUBJECT TO CHANGE IN FUTURE FEATURE DESIGN STUDIES. THE AREAS WITHIN THE PROPOSED RIGHTS-OF-WAY INCLUDE FEE ACQUISITIONS, TEMPORARY CONSTRUCTION EASEMENTS, PERMANENT ROAD AND FLOWAGE EASEMENTS. (ALSO SEE PLATE 28 FOR FEE ACQUISITION BIOLOGICAL MITIGATION LANDS.)

2-EXISTING BEAR VALLEY HIGH LINE AQUEDUCT AND THE GREENSPOT PIPELINE TO BE PROTECTED IN PLACE OR AVOIDED INSIDE THE IMPERVIOUS BORROW AREA. ALSO SEE PLATES 25, 26 AND GEOTECHNICAL APPENDIX A, PLATE A-60, FOR FURTHER DETAILS.

3-EXISTING WATER PERCOLATION BASINS AND DIVERSION FACILITIES TO BE REPLACED. SEE GEOTECHNICAL APPENDIX A, PLATE A-101, FOR DETAILS.

SOURCE OF INFORMATION

U.S. GEOLOGICAL SURVEY 7.5 MINUTE SERIES QUADRANGLES OF REDLANDS AND YUCAIPA, CALIFORNIA, DATED 1967, PHOTO REVISED 1980

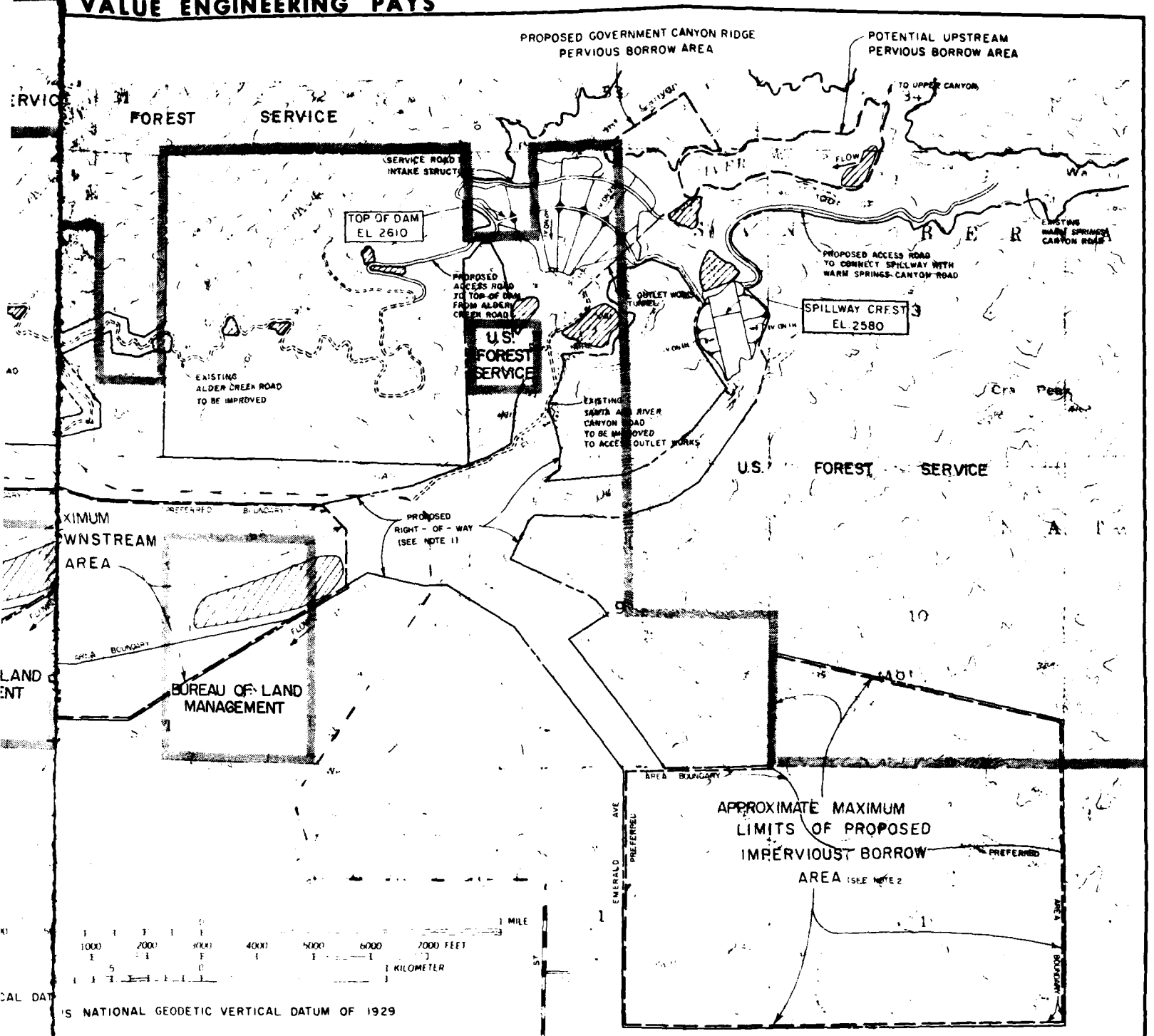
LEGEND

PROPOSED DISPOSAL SITE OR STOCKPILE AREA

APPROXIMATE BOUNDARY OF FEDERAL LAND (U.S. FOREST SERVICE OR BUREAU OF LAND MANAG

PROPOSED RIGHT-OF-WAY (SEE NOTE 1)

VALUE ENGINEERING PAYS



SYMBOL	DESCRIPTION	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY: QC
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

PRELIMINARY LAND REQUIREMENTS AND OWNERSHIP MAP

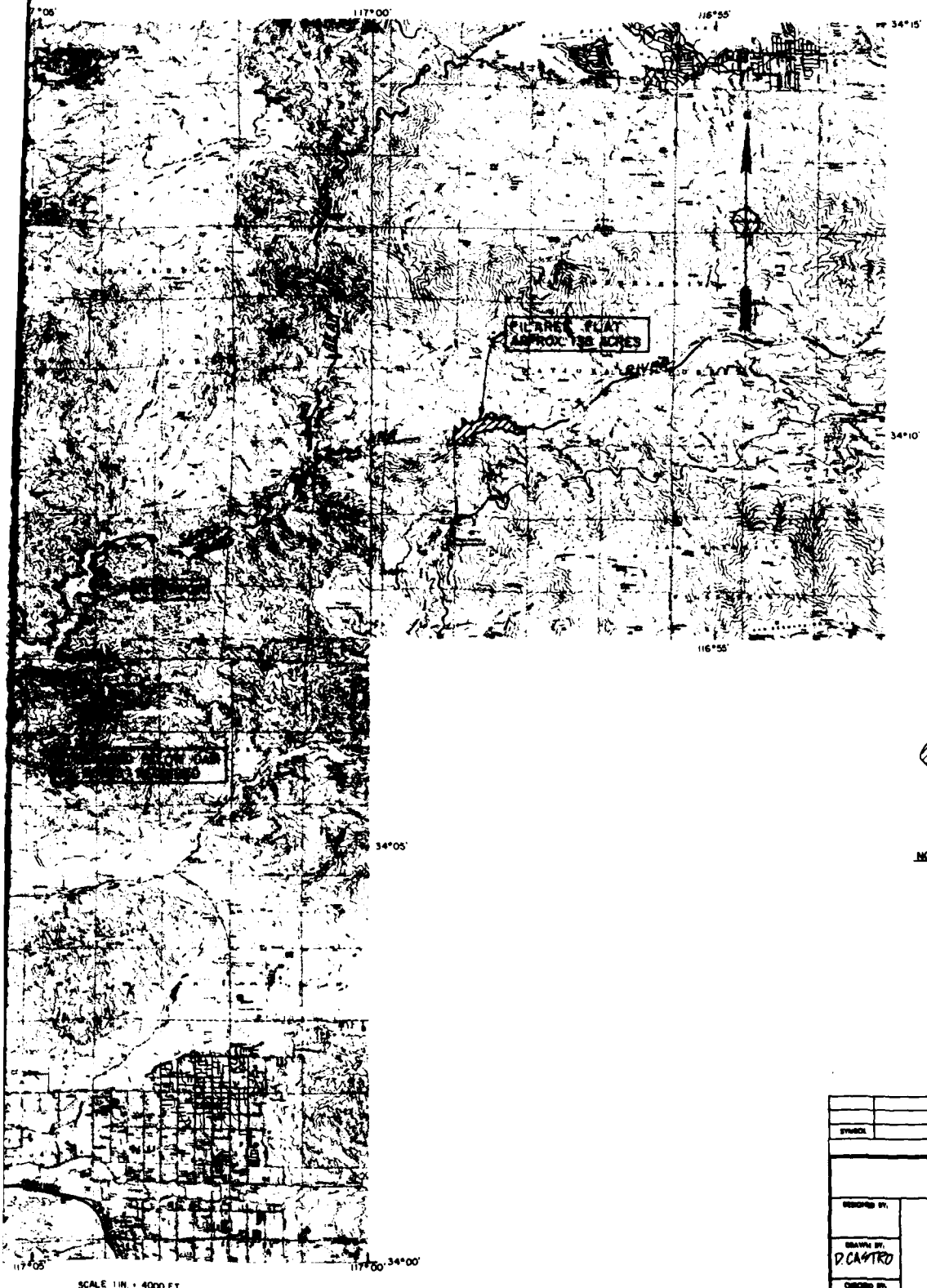
SUBMITTED BY: DATE APPROVED:
SPEC. NO. DACW09- - -
DISTRICT FILE NO.
SHEET

PLATE 2A

SAFETY PAYS



ENVIRONMENTAL
ENHANCEMENT
THRU ENGINEERING



LEGEND

BIOLOGICAL MITIGATION LANDS TO BE ACQUIRED

NOTES.

1. MITIGATION LAND BOUNDARIES ARE APPROXIMATED
2. LANDS REQUIRED TO MITIGATE THE IMPACTS TO THE ERIASTRUM PLANTS WILL BE LOCATED IN THE SANTA ANA WASH EXACT AREA AND ACREAGE WILL BE FINALIZED IN FUTURE STUDIES

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:

DRAWN BY: D. CASTRO

CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

BIOLOGICAL MITIGATION LANDS

SUBMITTED BY: DATE APPROVED:

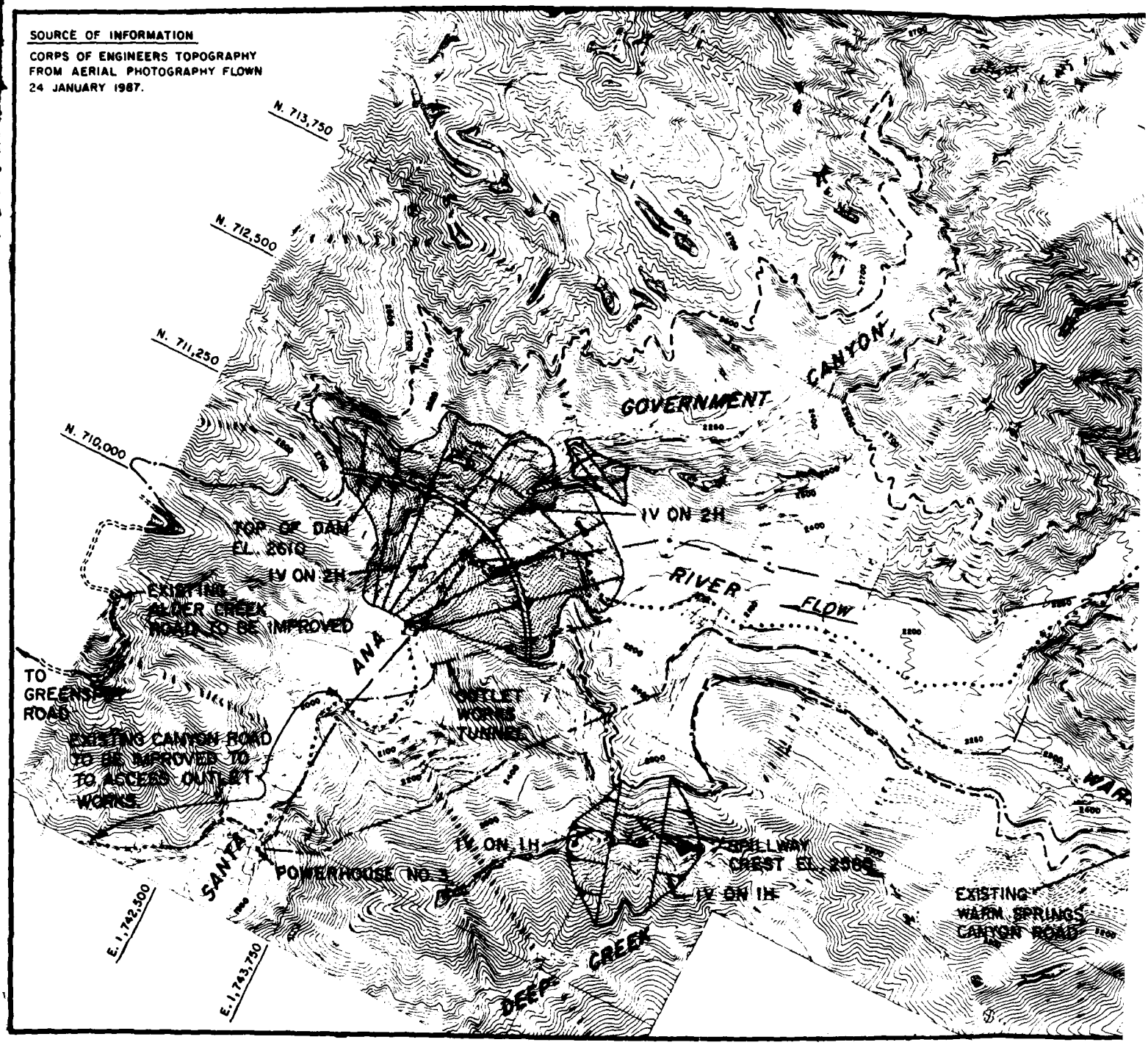
CHIEF, ENGINEERING DIVISION

SPEC. NO. DACW09- ____ B- ____

DISTRICT FILE NO.

SHEET

PLATE 28


SOURCE OF INFORMATION
CORPS OF ENGINEERS TOPOGRAPHY
FROM AERIAL PHOTOGRAPHY FLOWN
24 JANUARY 1987.
N. 713,750
N. 712,500
N. 711,250
N. 710,000
GOVERNMENT
CANYON
TOP OF DAM
EL. 2610
1V ON 2H
EXISTING
ALDER CREEK
ROAD TO BE IMPROVED
RIVER
FLOW
ANA
TO
GREENS
ROAD
EXISTING CANYON ROAD
TO BE IMPROVED TO
TO ACCESS OUTLET
WORKS
OUTLET
WORKS
TUNNEL
SANTA
POWERHOUSE NO. 3
1V ON 1H
SPILLWAY
CREST EL. 2586
1V ON 1H
EXISTING
WARM SPRINGS
CANYON ROAD
DEEP
CREEK
E. 1,742,500
E. 1,743,750

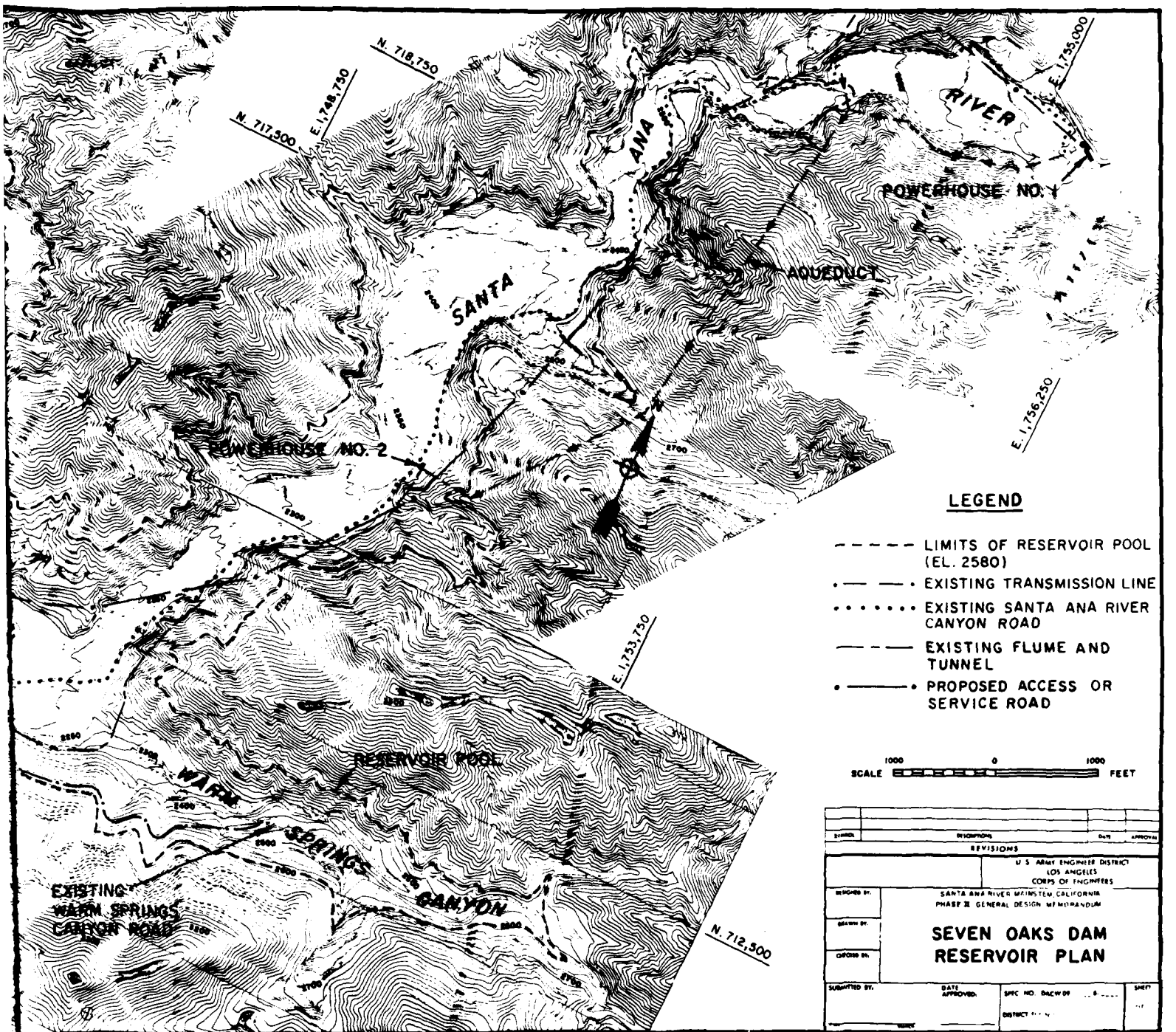
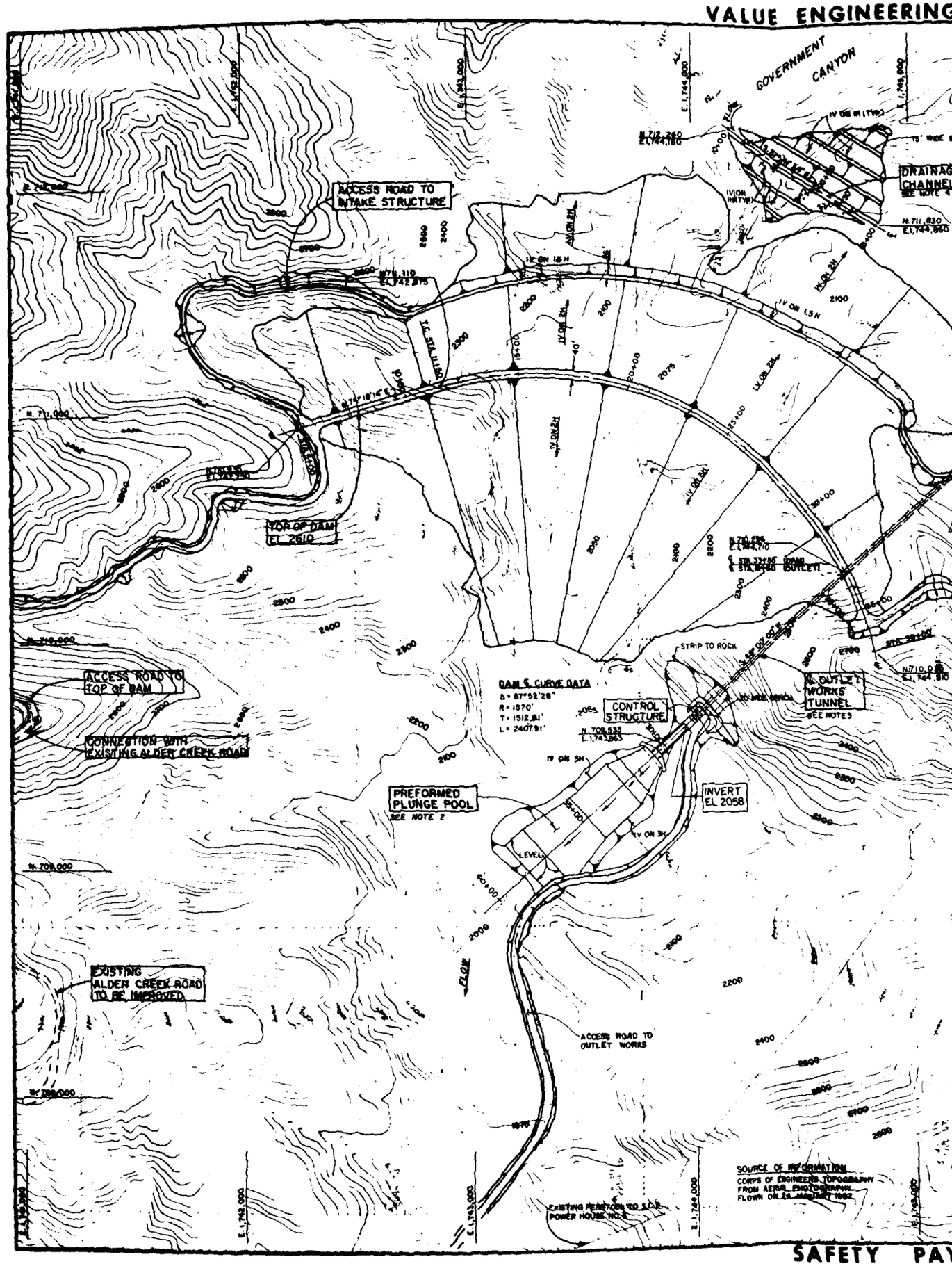
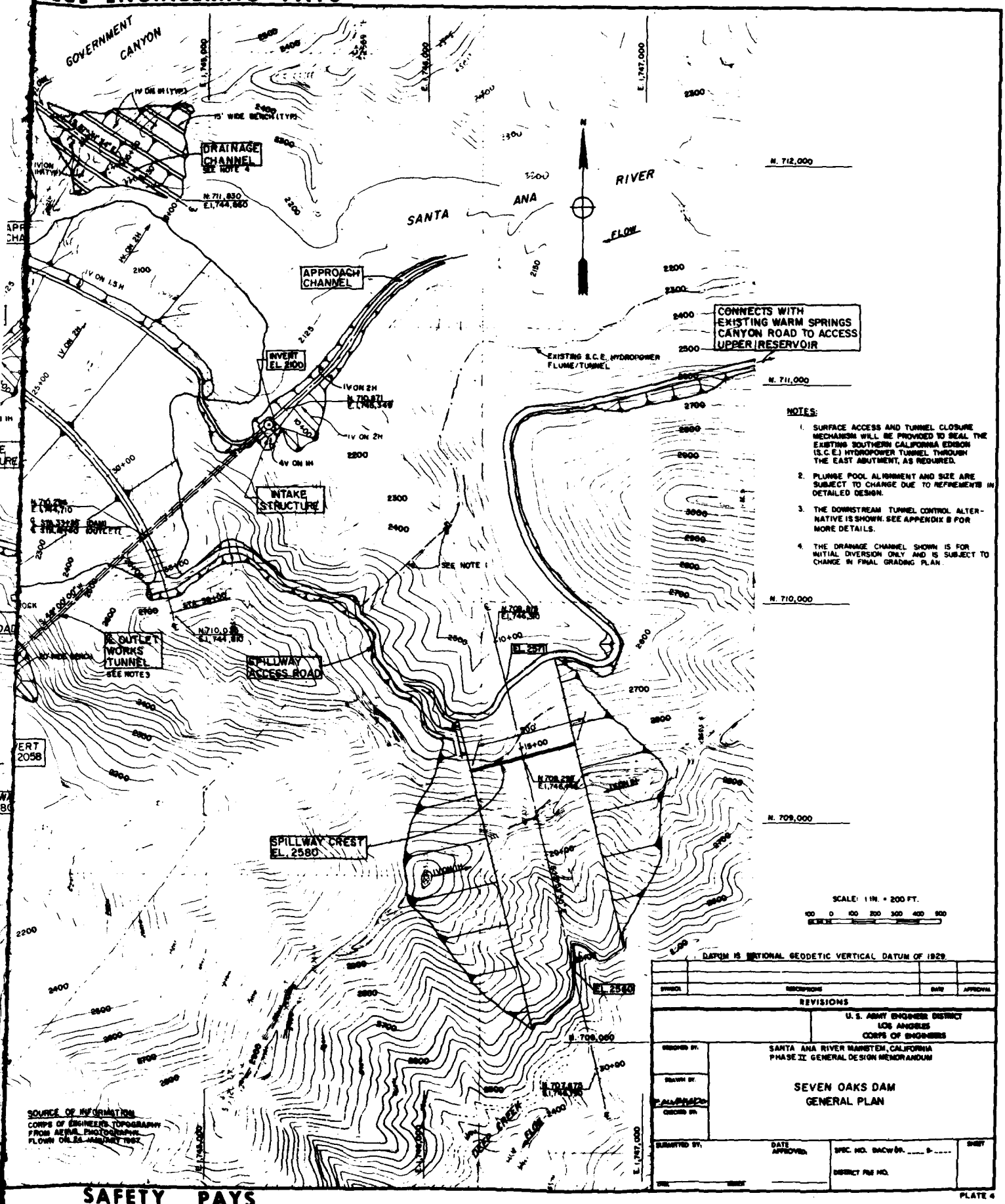

N. 718,750
N. 717,300
E. 1,748,750
E. 1,755,000
SANTA ANA RIVER
POWERHOUSE NO. 1
AQUEDUCT
E. 1,756,250
POWERHOUSE NO. 2
E. 1,753,750
RESERVOIR POOL
WARM SPRINGS CANYON
EXISTING WARM SPRINGS CANYON ROAD
N. 712,500
LEGEND
LIMITS OF RESERVOIR POOL (EL. 2580)
EXISTING TRANSMISSION LINE
EXISTING SANTA ANA RIVER CANYON ROAD
EXISTING FLUME AND TUNNEL
PROPOSED ACCESS OR SERVICE ROAD
1000 0 1000
SCALE FEET
REVISIONS
U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS
SANTA ANA RIVER MAINSTEM CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM
SEVEN OAKS DAM
RESERVOIR PLAN

PLATE 3


GOVERNMENT CANYON
ACCESS ROAD TO INTAKE STRUCTURE
DRAINAGE CHANNEL
TOP OF DAM EL. 2610
ACCESS ROAD TO TOP OF DAM
CONNECTION WITH EXISTING ALDER CREEK ROAD
EXISTING ALDER CREEK ROAD TO BE IMPROVED
PREFORMED PLUNGE POOL
SEE NOTE 2
DAM & CURVE DATA
Δ = 87°52'28"
R = 1570'
T = 1512.81'
L = 2407.91'
CONTROL STRUCTURE
OUTLET WORKS TUNNEL
SEE NOTE 3
INVERT EL 2058
STRIP TO ROCK
ACCESS ROAD TO OUTLET WORKS
EXISTING PENSTOCK TO S.C.E. POWER HOUSE NO. 8
SOURCE OF INFORMATION
CORPS OF ENGINEERS TOPOGRAPHY FROM AERIAL PHOTOGRAPHS FLOWN ON 25 JANUARY 1962



NOTES:

1. SURFACE ACCESS AND TUNNEL CLOSURE MECHANISM WILL BE PROVIDED TO SEAL THE EXISTING SOUTHERN CALIFORNIA EDISON (S.C.E.) HYDROPOWER TUNNEL THROUGH THE EAST ABUTMENT, AS REQUIRED.
2. PLUNGE POOL ALIGNMENT AND SIZE ARE SUBJECT TO CHANGE DUE TO REFINEMENTS IN DETAILED DESIGN.
3. THE DOWNSTREAM TUNNEL CONTROL ALTERNATIVE IS SHOWN. SEE APPENDIX B FOR MORE DETAILS.
4. THE DRAINAGE CHANNEL SHOWN IS FOR INITIAL DIVERSION ONLY AND IS SUBJECT TO CHANGE IN FINAL GRADING PLAN.

SOURCE OF INFORMATION
CORPS OF ENGINEERS TOPOGRAPHY FROM AERIAL PHOTOGRAPHS FLOWN ON 24 JANUARY 1982.

SYMBOL	DESCRIPTION	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

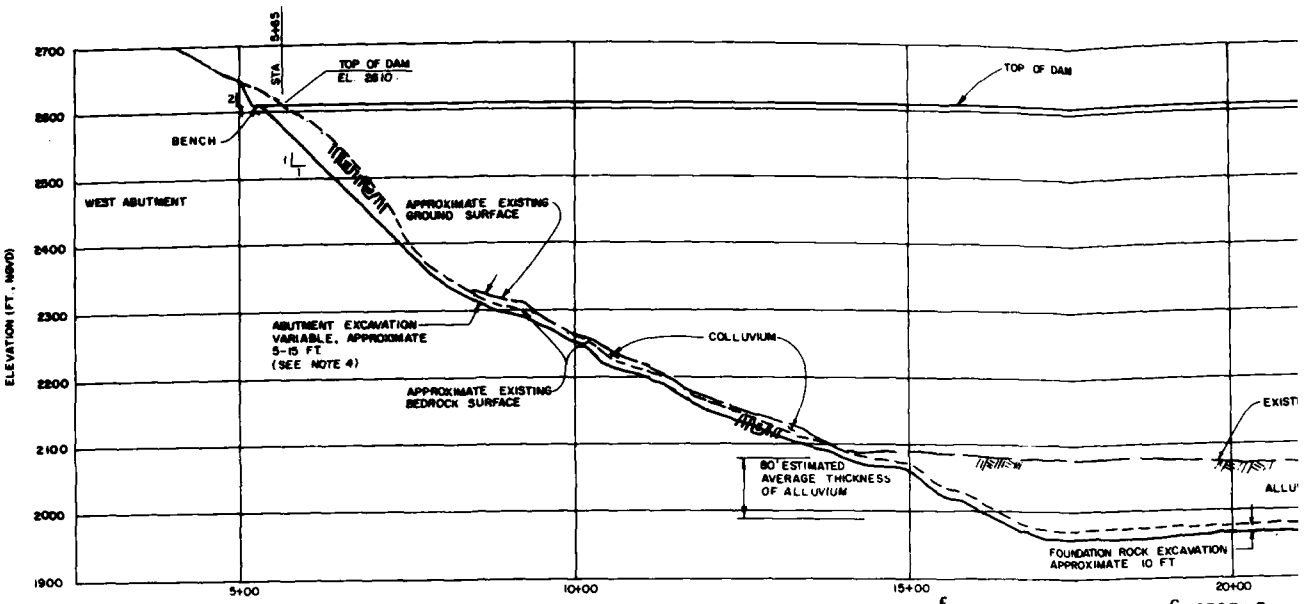
DESIGNED BY:
DRAWN BY:
TRACED BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

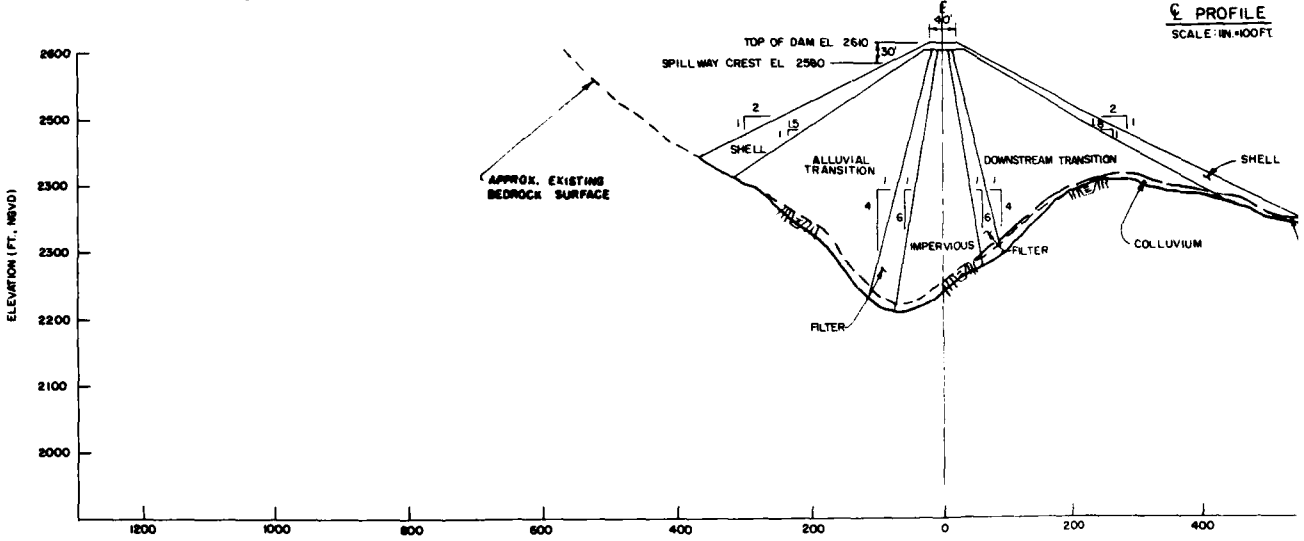
SEVEN OAKS DAM
GENERAL PLAN

SUBMITTED BY:
DATE APPROVED:
SPEC. NO. DACW09-____-____
DISTRICT FILE NO.
SHEET

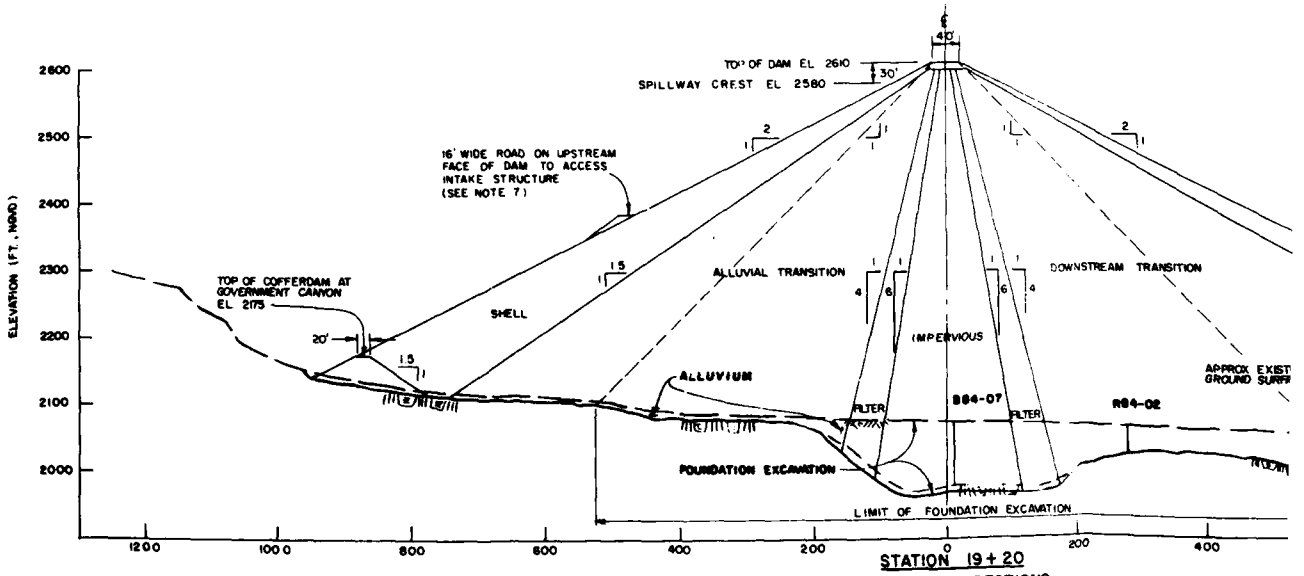
PLATE 4



℄ PROFILE
SCALE: 1IN.=100FT.



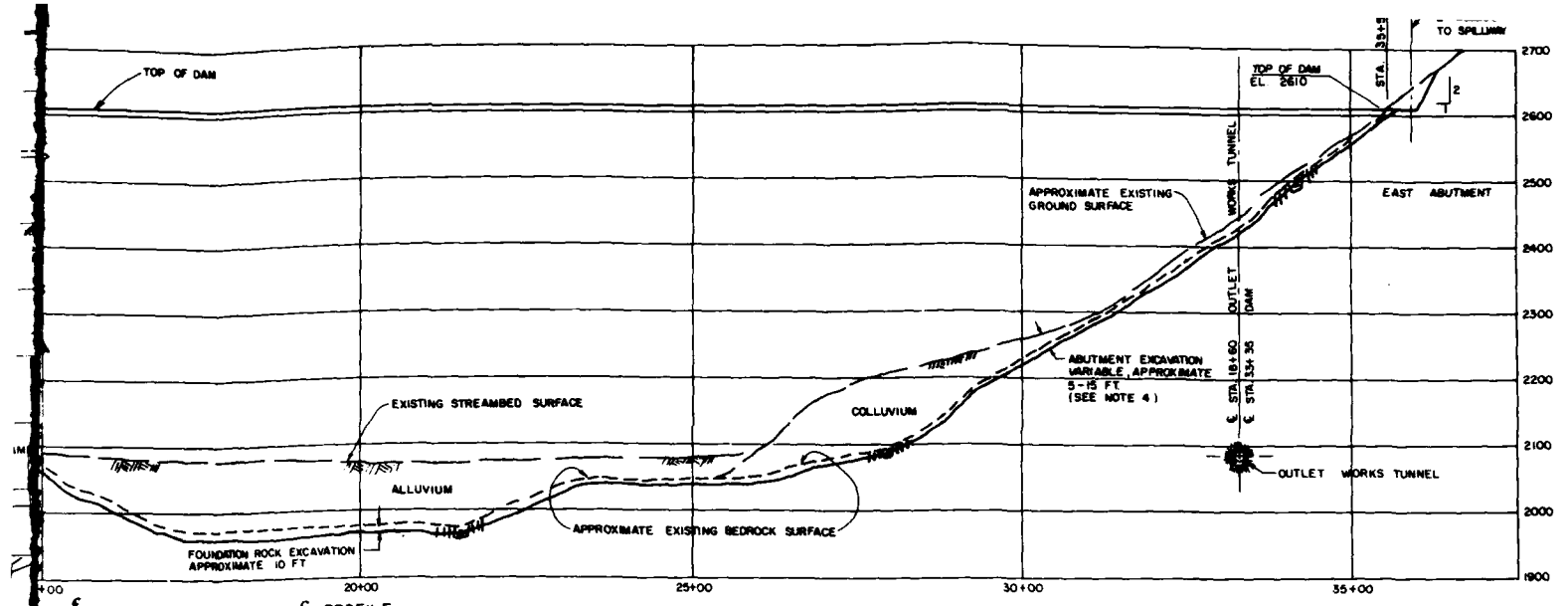
STATION 10+00



STATION 19+20

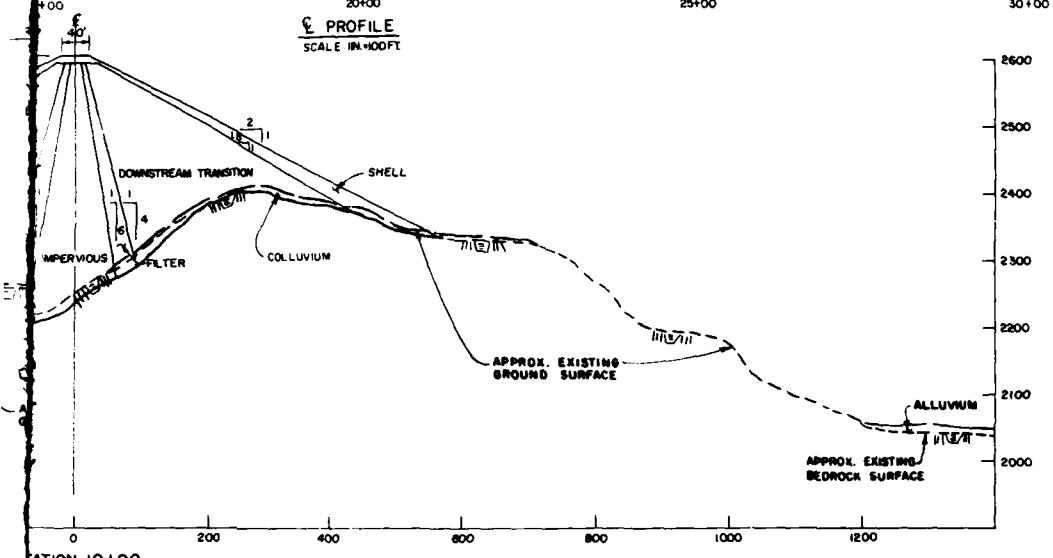
EMBANKMENT SECTIONS
SCALE: 1IN.=100FT.

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

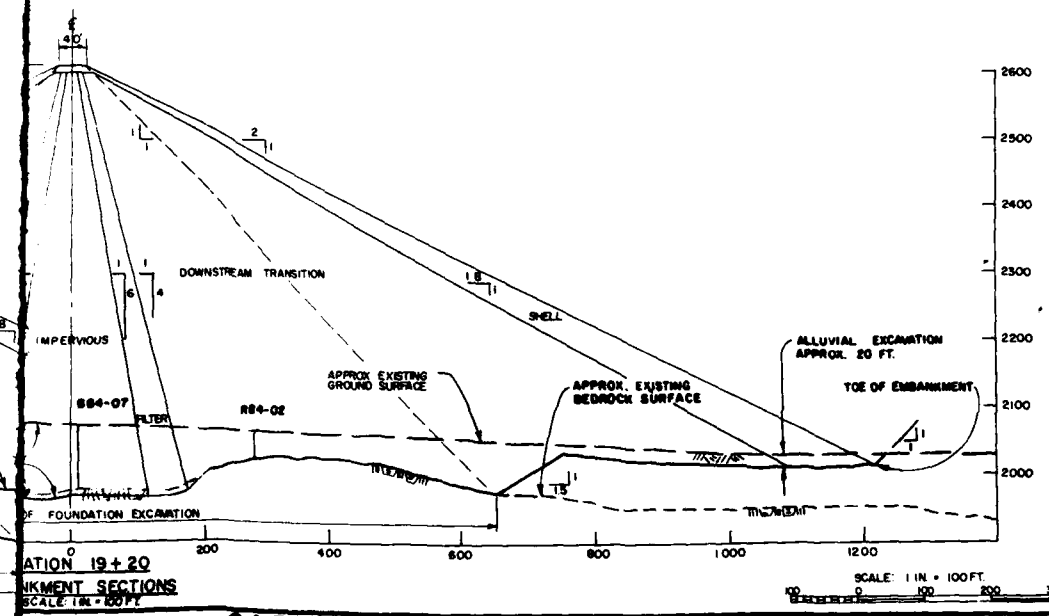


℄ PROFILE

SCALE IN.=100FT.



ATION 10+00



ATION 19+20

NKMENT SECTIONS

SCALE: 1 IN. = 100 FT.

SCALE: 1 IN. = 100 FT.

-100 0 100 200 300

NOTES:

1. ℄ PROFILE IS DRAWN LOOKING UPSTREAM.
2. EMBANKMENT SECTIONS ARE DRAWN LOOKING EAST.
3. BEDROCK SURFACE APPROXIMATED AND SMOOTHED.
4. FINAL CONFIGURATION AND DEPTH OF ABUTMENT EXCAVATION WILL BE DETERMINED IN DETAILED DESIGN.
5. B84-07, R84-02 DENOTE GEOTECHNICAL BECKER AND ROTARY DRILLINGS, YEAR DRILLED AND HOLE NUMBER.
6. SEE GEOTECHNICAL APPENDIX A, PLATE A-80, FOR EMBANKMENT MATERIAL GRADATIONS AND UPSTREAM COFFERDAM DETAIL.
7. SEE PLATE 21 FOR ACCESS ROAD DETAILS.

DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:

DRAWN BY: DE

CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

EMBANKMENT PROFILE AND SECTIONS

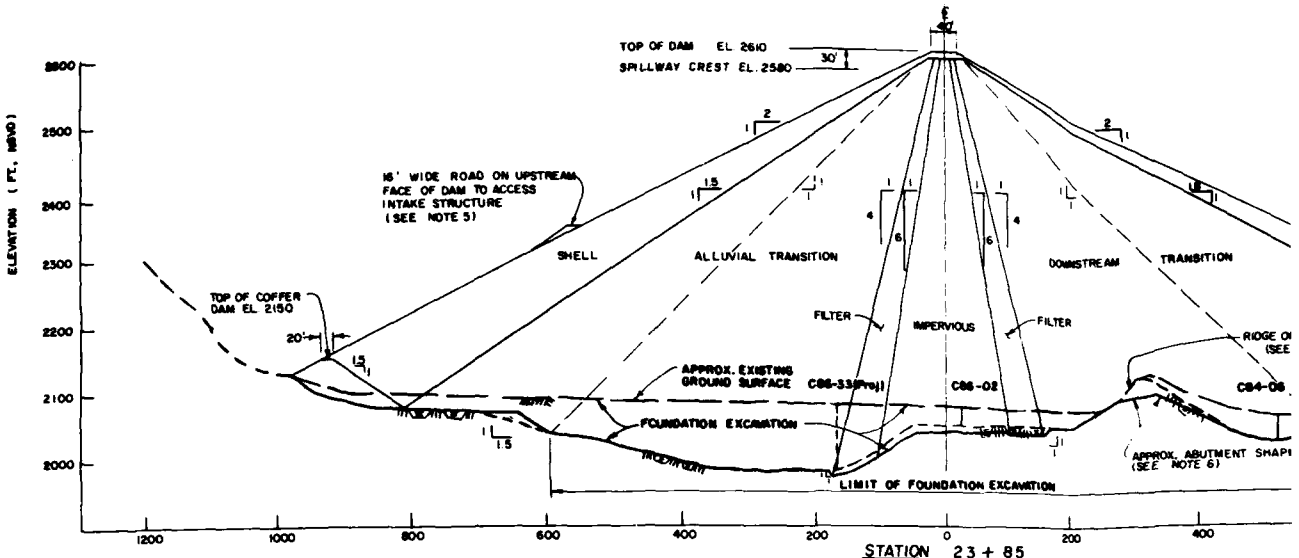
SUBMITTED BY: DATE APPROVED:

DISTRICT FILE NO.

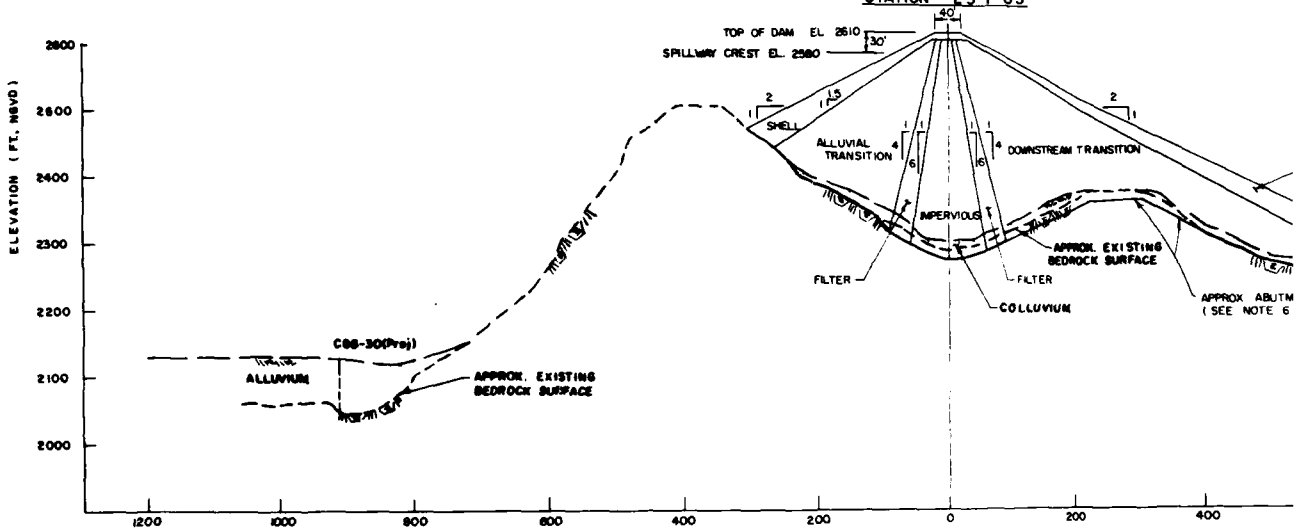
SHEET

PLATE

SAFETY PAYS



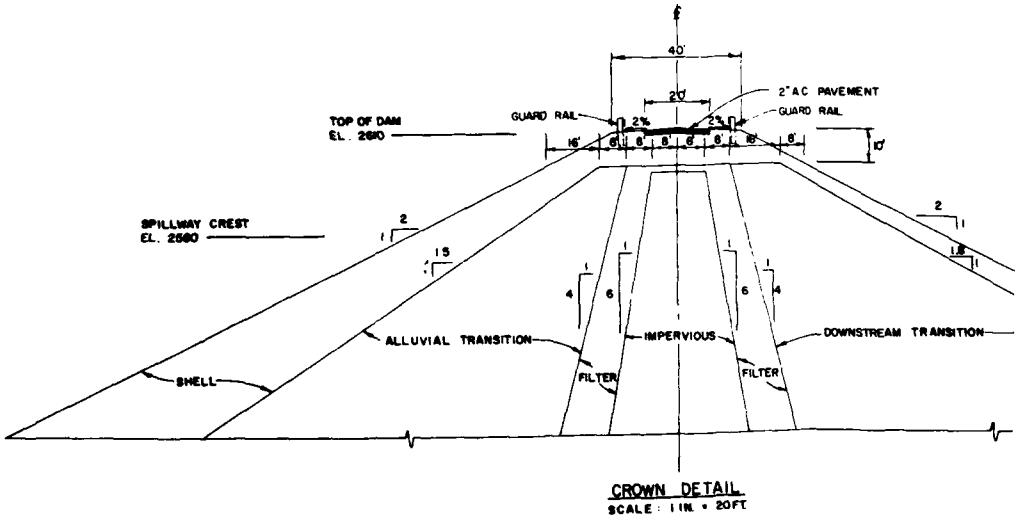
STATION 23+85



STATION 31+00

EMBANKMENT SECTIONS

SCALE 1 IN = 100 FT



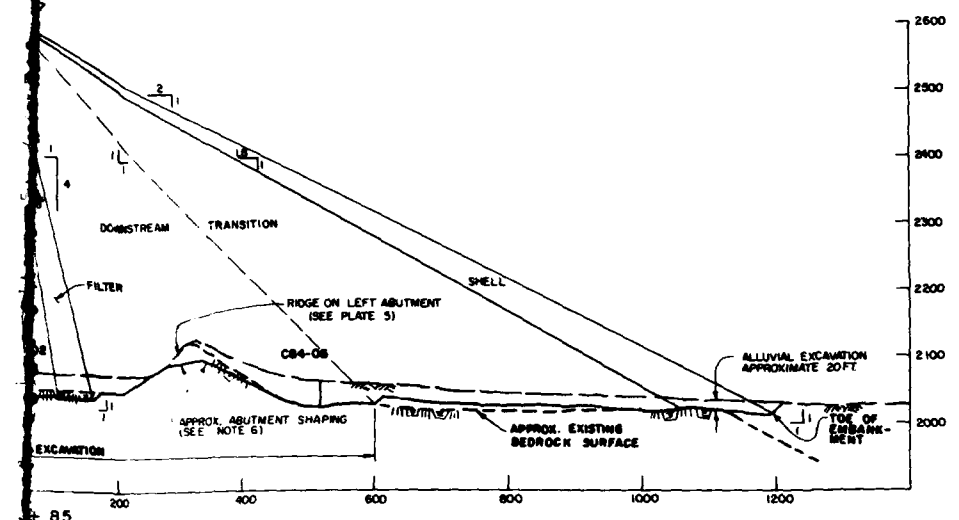
CROWN DETAIL

SCALE : 1 IN. = 20 FT

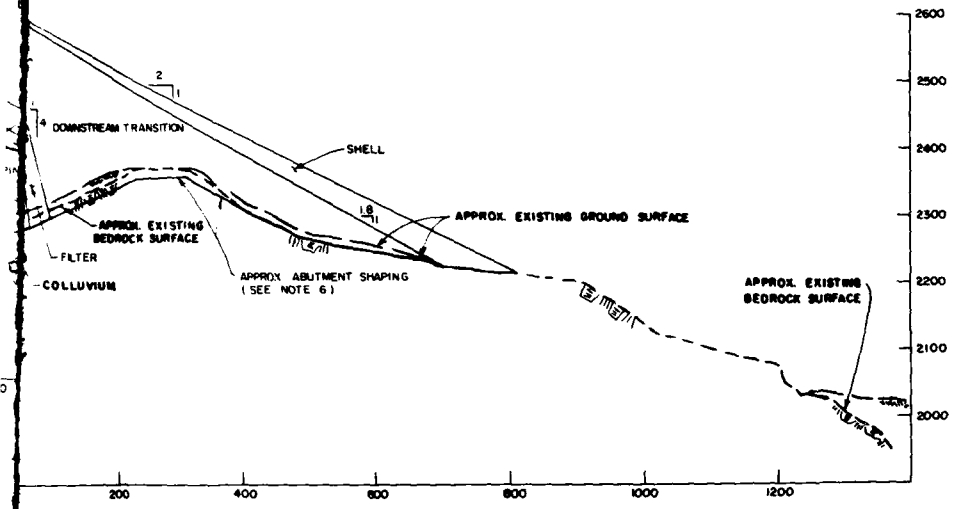
ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

SAFETY

VALUE ENGINEERING PAYS



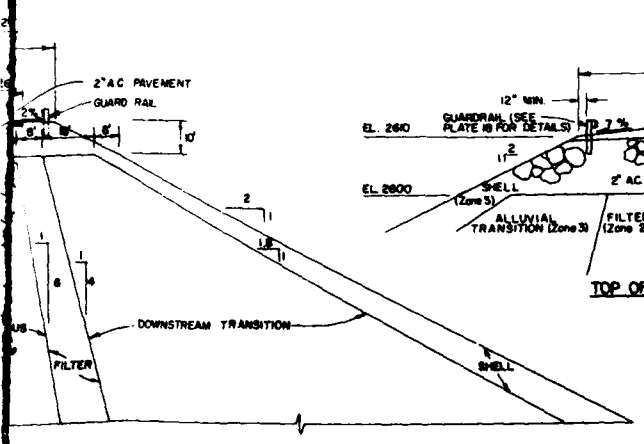
+85



+00

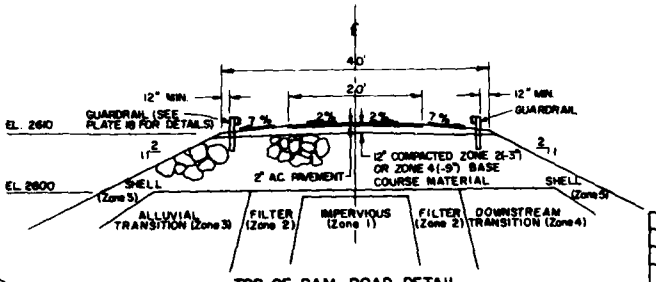
SECTIONS

100 FT



TAIL

= 20FT



TOP OF DAM ROAD DETAIL
NOT TO SCALE

SCALE: 1 IN. = 20 FT.
10 0 10 20 30 40 50

SCALE: 1 IN. = 100 FT.
100 0 100 200 300

NOTES:

1. EMBANKMENT SECTIONS ARE DRAWN LOOKING EAST.
2. BEDROCK SURFACE APPROXIMATED AND SMOOTHED.
3. C86-02 DENOTES GEOTECHNICAL CORE HOLE, YEAR DRILLED AND HOLE NUMBER
4. SEE GEOTECHNICAL APPENDIX A, PLATE A-80, FOR EMBANKMENT MATERIAL GRADATIONS AND UPSTREAM COFFERDAM DETAILS.
5. SEE PLATE 21 FOR ACCESS ROAD DETAILS.
6. REFINEMENT DETAILS TO ABUTMENT SHAPING WILL BE SHOWN IN FUTURE FEATURE DESIGN STUDIES

DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY: DC
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

EMBANKMENT SECTIONS AND DETAILS

SUBMITTED BY: DATE APPROVED

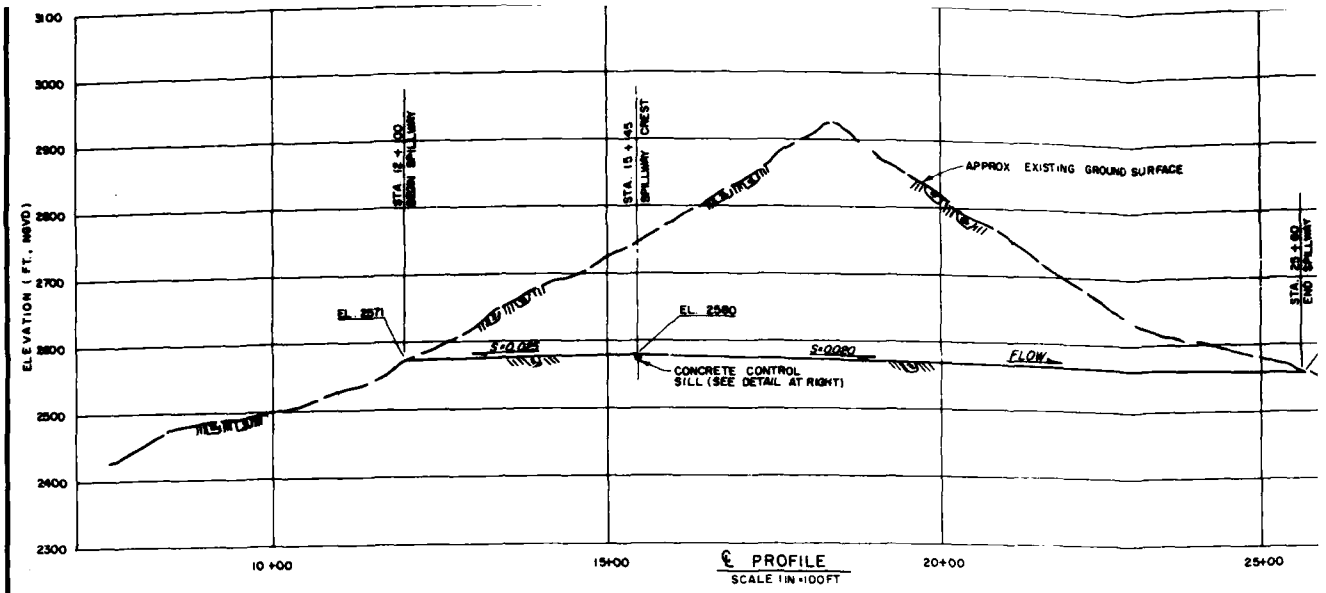
SPEC. NO. DACW09- ____ B- ____

DISTRICT FILE NO.

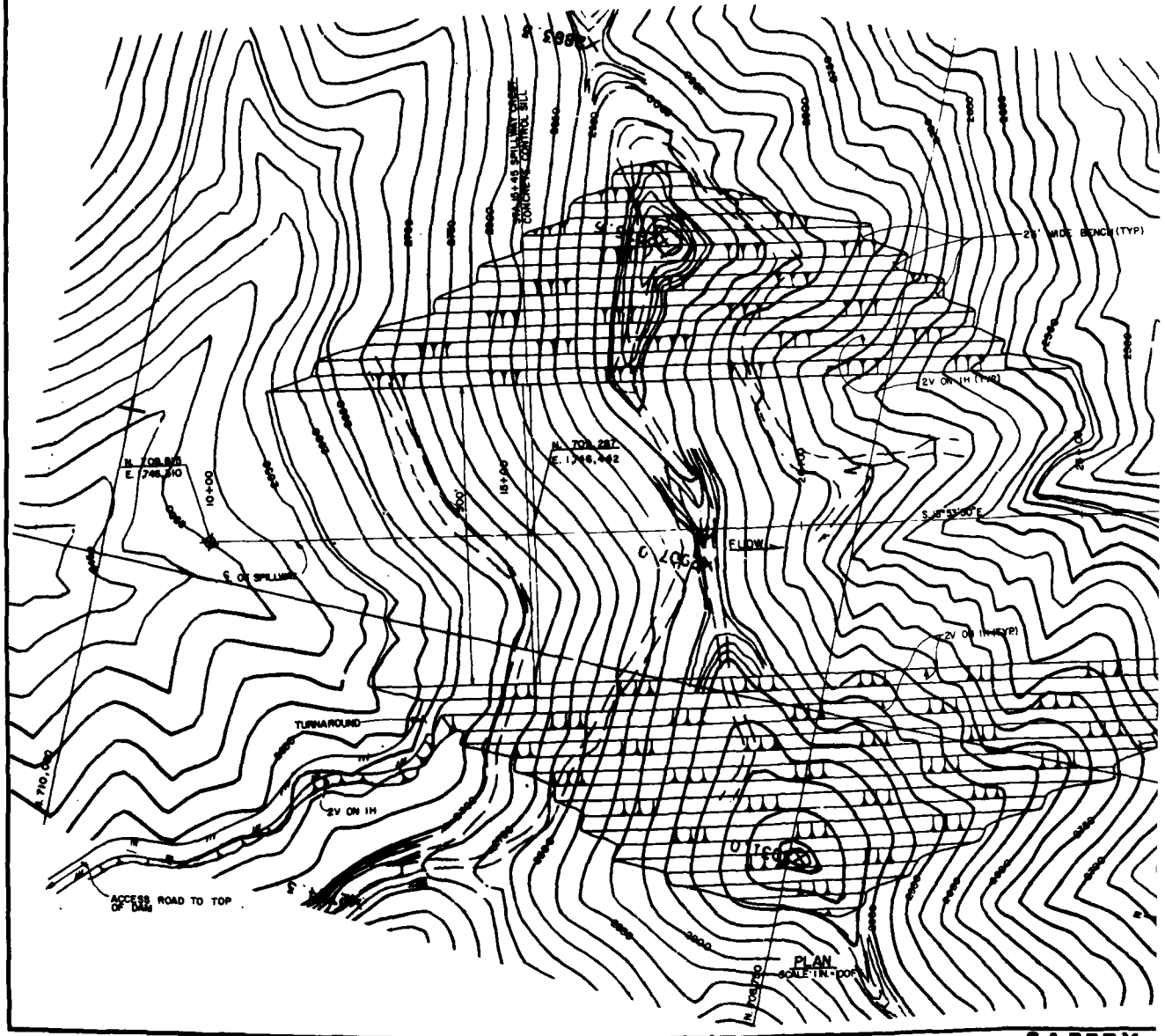
SHEET

PLATE 6

SAFETY PAYS



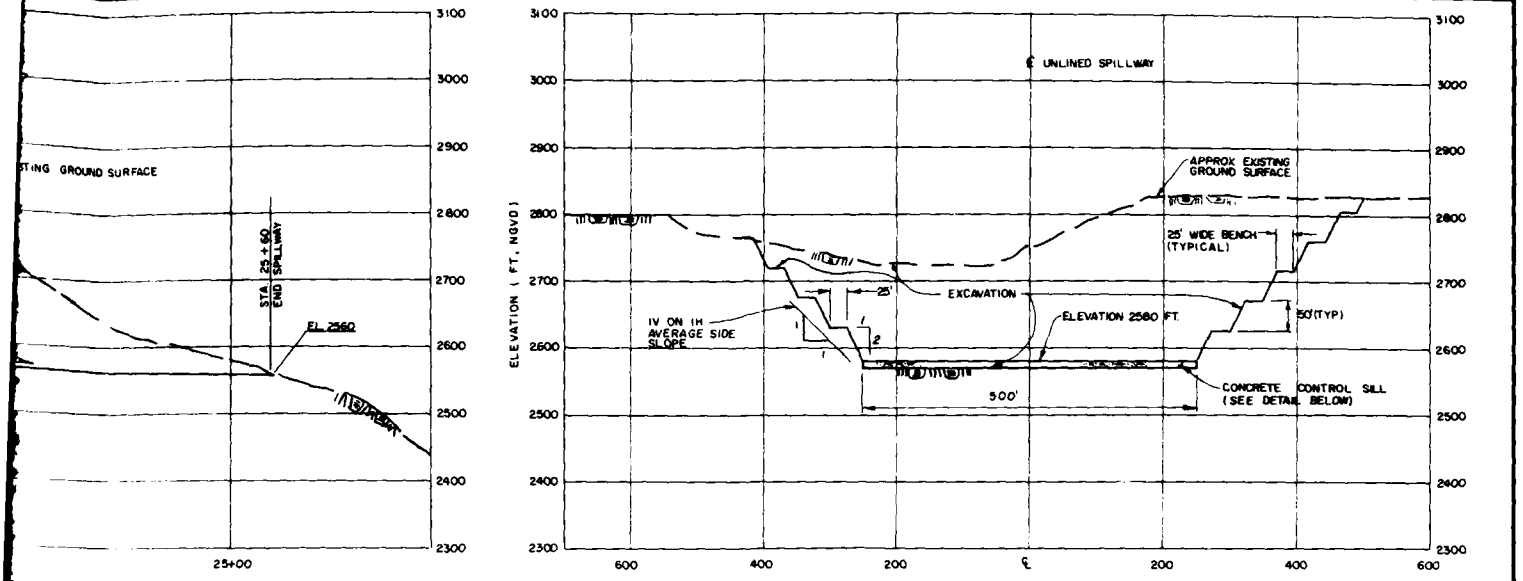
℄ PROFILE
SCALE 1 IN = 100 FT



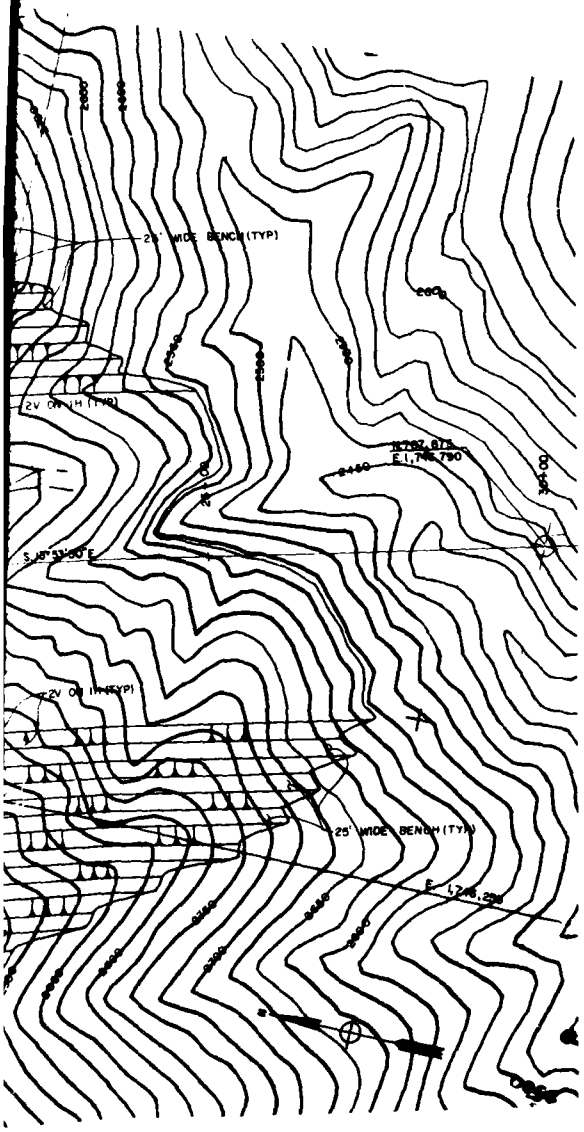
ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

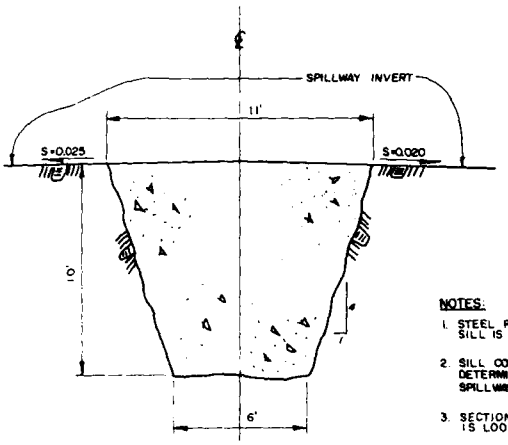
SAFETY

VALUE ENGINEERING PAYS



SECTION AT SPILLWAY CREST
SCALE 1 IN = 100 FT





CONCRETE CONTROL SILL DETAIL
NOT TO SCALE

NOTES:
1. STEEL REINFORCEMENT IN SILL IS NOT SHOWN
2. SILL CONFIGURATION TO BE DETERMINED AFTER SPILLWAY EXCAVATION.
3. SECTION AT SPILLWAY CREST IS LOOKING UPSTREAM.

SOURCE OF INFORMATION
CORPS OF ENGINEERS TOPOGRAPHY FROM AERIAL PHOTOGRAPHY FLOWN 24 JANUARY 1987.

SCALE 1 IN. = 100 FT
100 0 100 200 300

DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY: JC
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

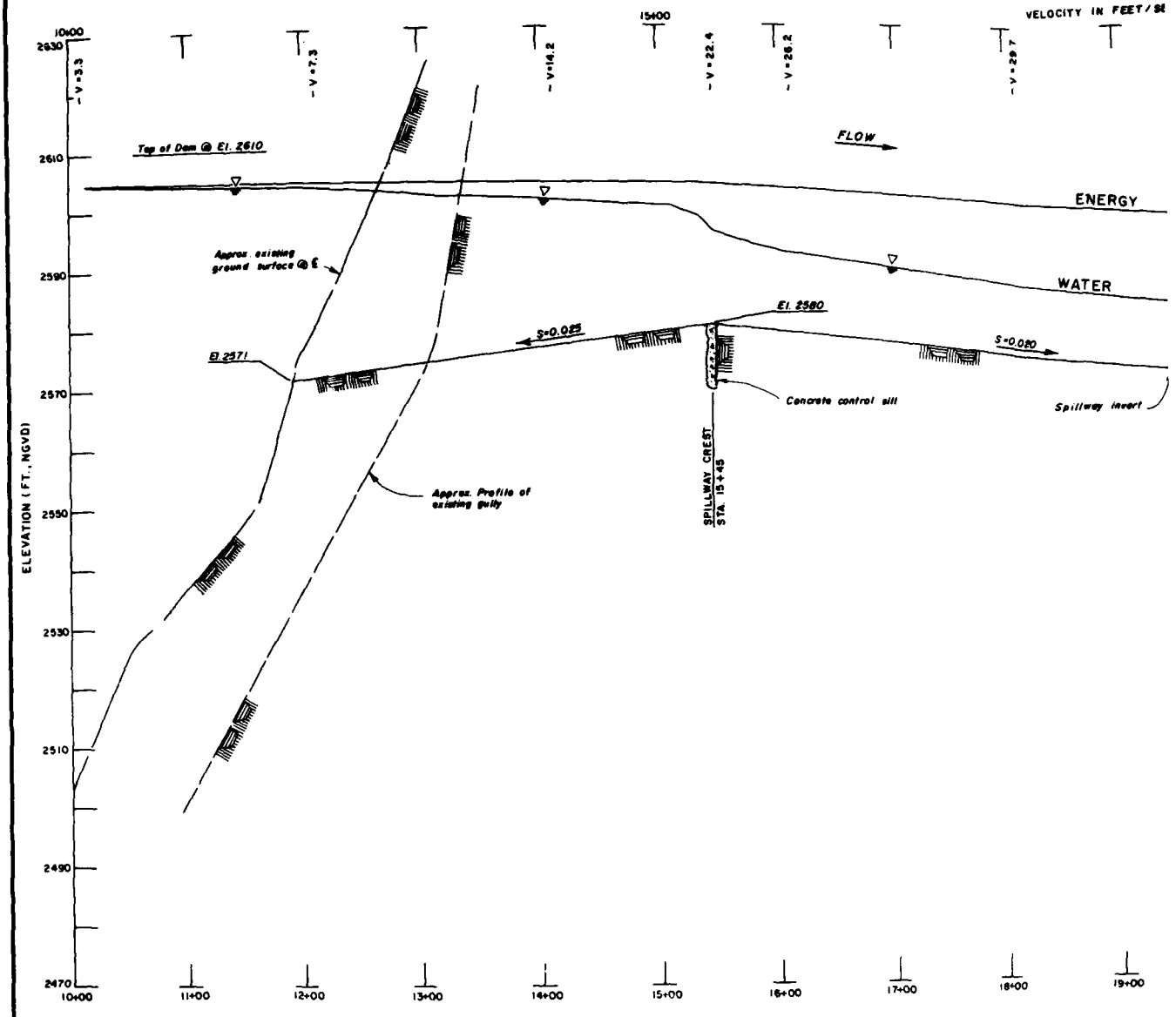
SEVEN OAKS DAM

SPILLWAY PLAN
PROFILE AND SECTIONS

SUBMITTED BY:
DATE APPROVED:
SPEC. NO. DACW09-____-____
DISTRICT FILE NO.
SHEET

PLATE

SAFETY PAYS



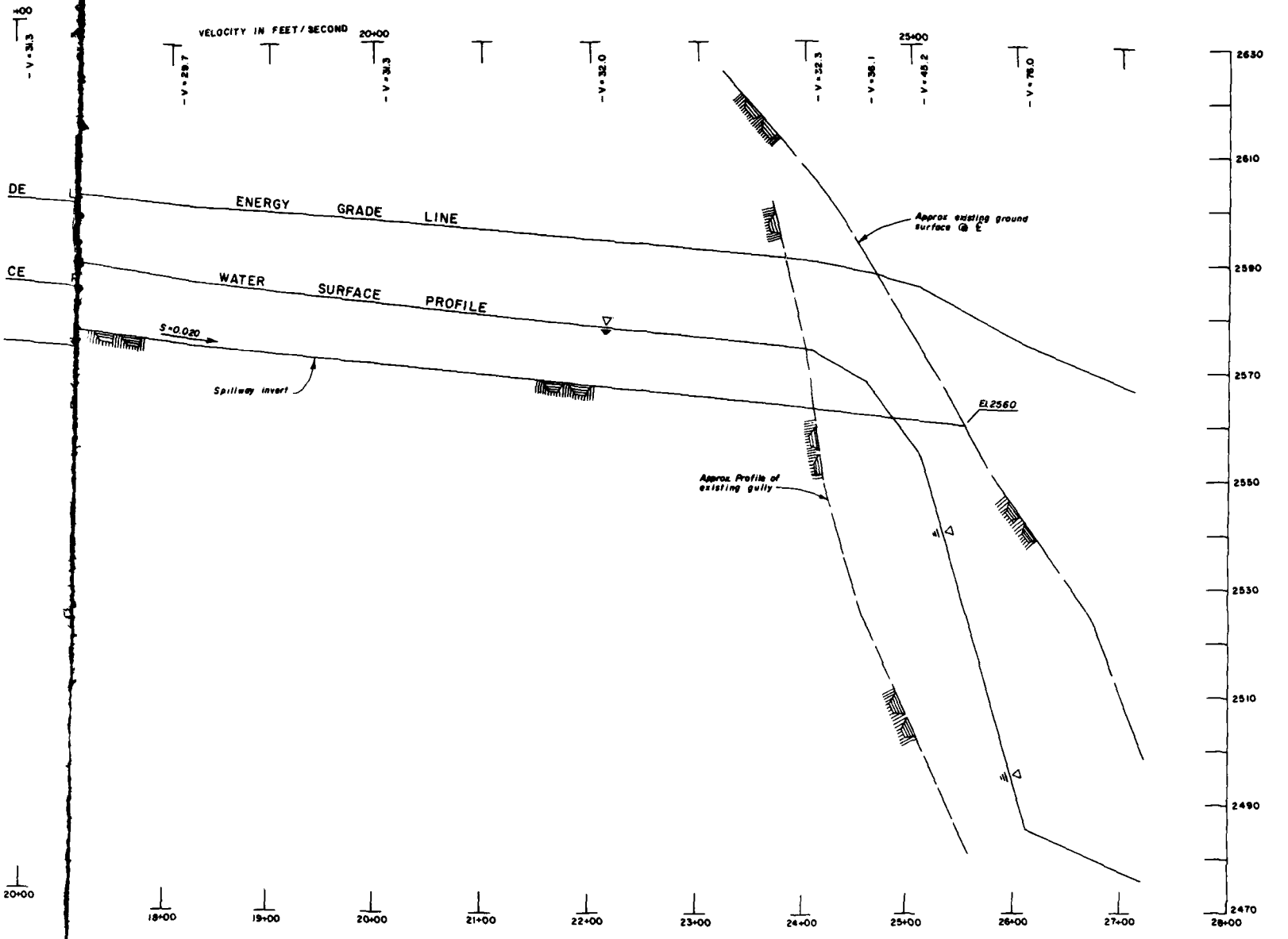
SPILLWAY HYDRAULIC PROFILE

HORIZ SCALE: 1 IN. = 50 FT.
VERT. SCALE: 1 IN. = 10 FT.

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

G P

VALUE ENGINEERING PAYS



ILLWAY HYDRAULIC PROFILE

HORIZ. SCALE: 1 IN. = 50 FT.
VERT. SCALE: 1 IN. = 10 FT.

SCALE: IN. = 50 FT.
50 0 50 100 150

SCALE: IN. = 10 FT.
10 0 10 20 30

DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

SYMBOL	DESCRIPTION	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY: KBA
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

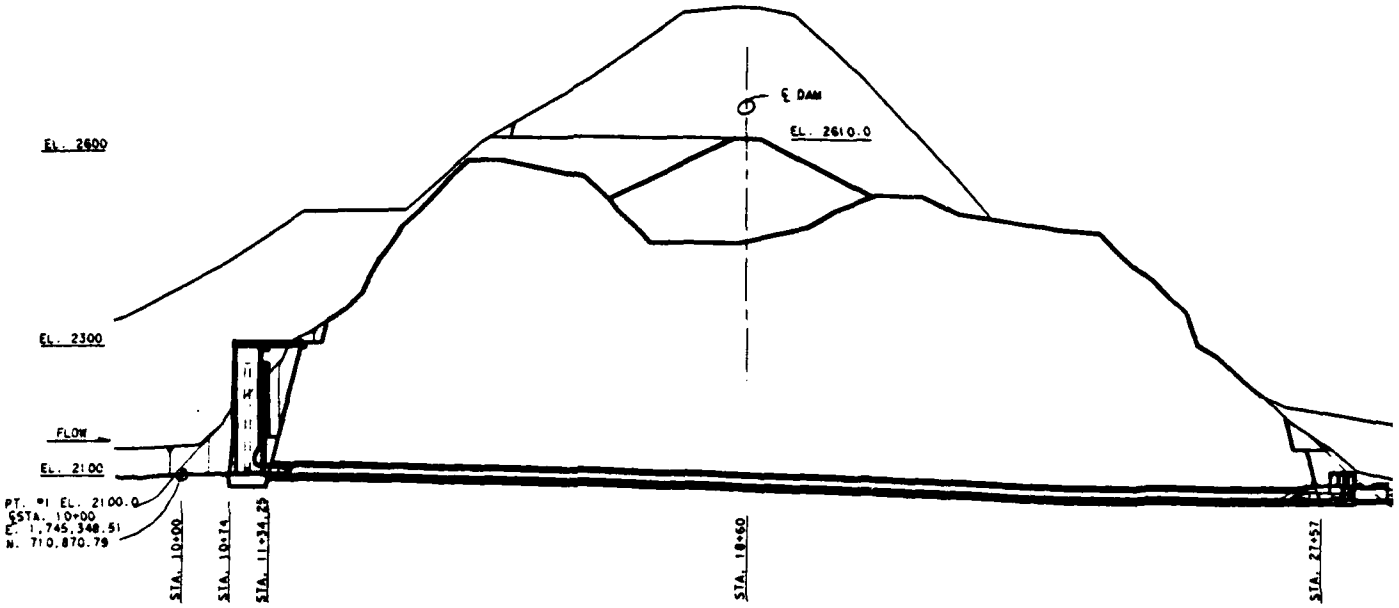
SEVEN OAKS DAM
SPILLWAY HYDRAULIC PROFILE

SUBMITTED BY: DATE APPROVED:
SPEC. NO. DACW09- -S-
DISTRICT FILE NO.
PLAT

AYS

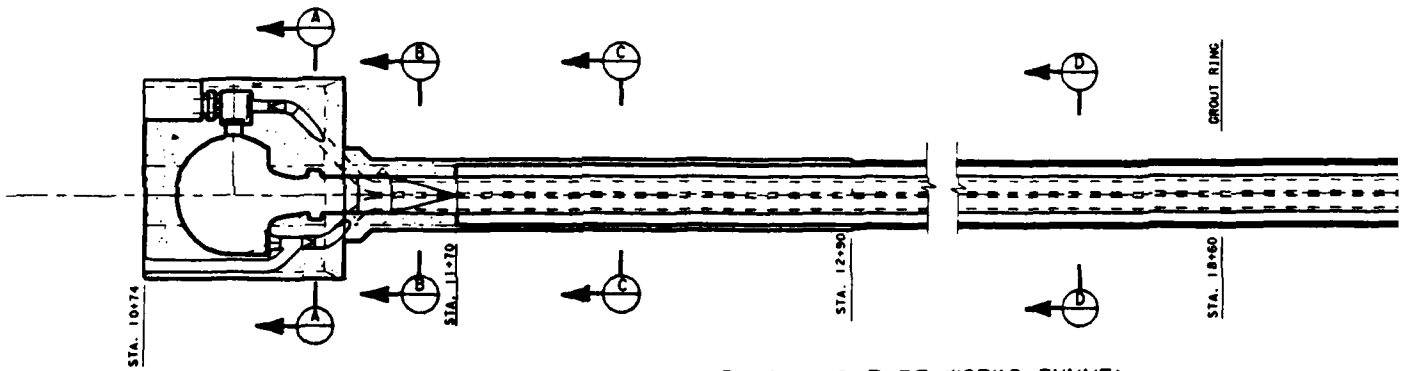
SAFETY PAYS

CORPS OF ENGINEERS



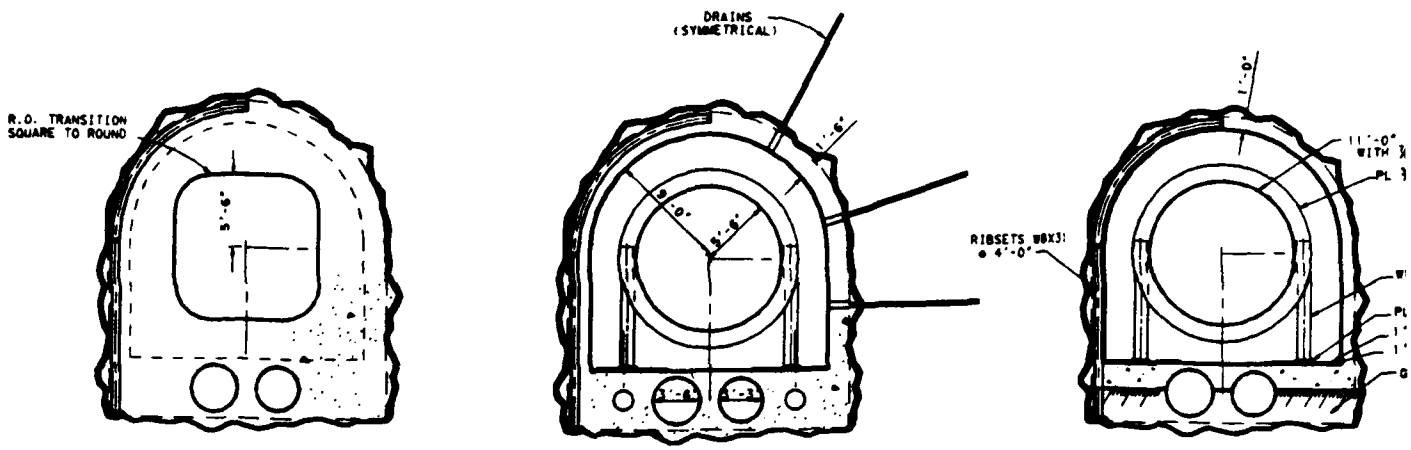
℄ PROFILE - OUTLET WORKS TUNNEL

SCALE: 1" = 100'



PLAN - OUTLET WORKS TUNNEL

SCALE: 1" = 20'



SECTION B

SCALE: 1" = 5'

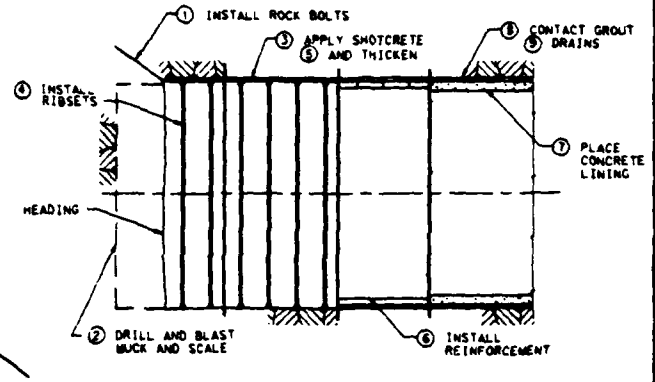
SECTION C

SCALE: 1" = 5'

SECTION D

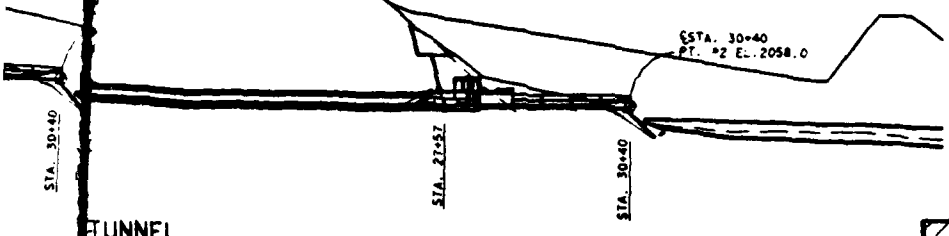
SCALE: 1" = 5'

U.S. ARMY

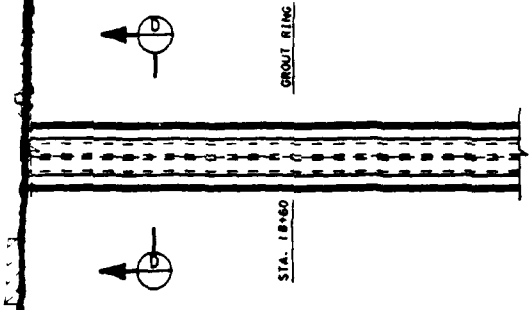


TUNNELING SEQUENCE

N.T.S.

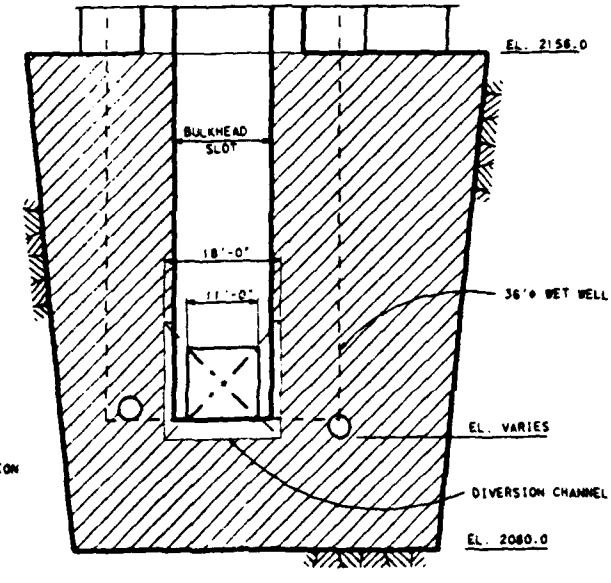


TUNNEL

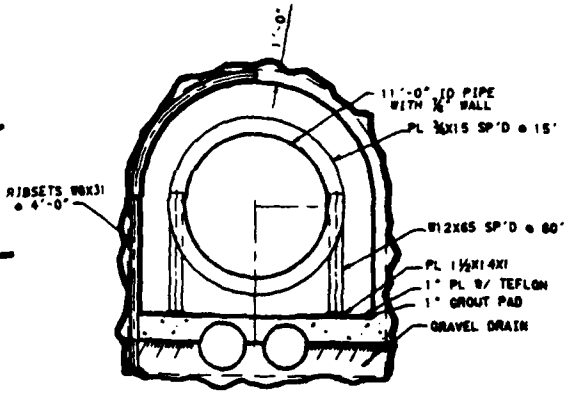


WORKS TUNNEL

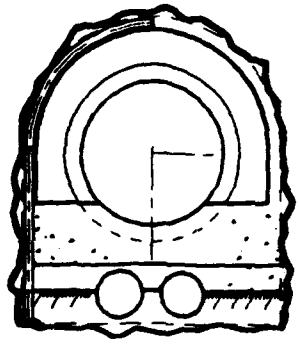
TUNNEL CONTRACT - FIRST STAGE CONSTRUCTION



SECTION (A)
SCALE: F = 10'



SECTION (D)
SCALE: F = 5'



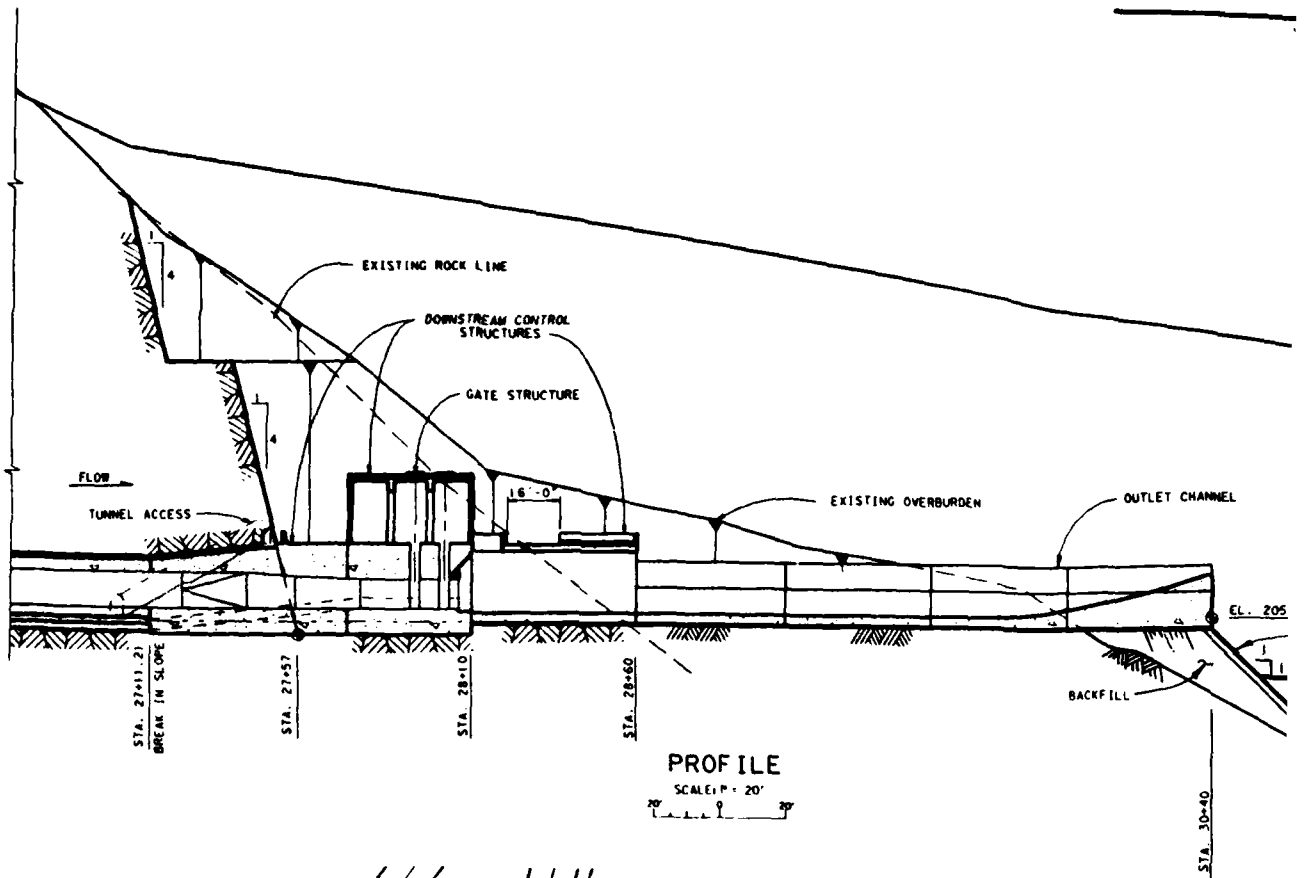
ALTERNATIVE CONDUIT SUPPORT

NOTE:

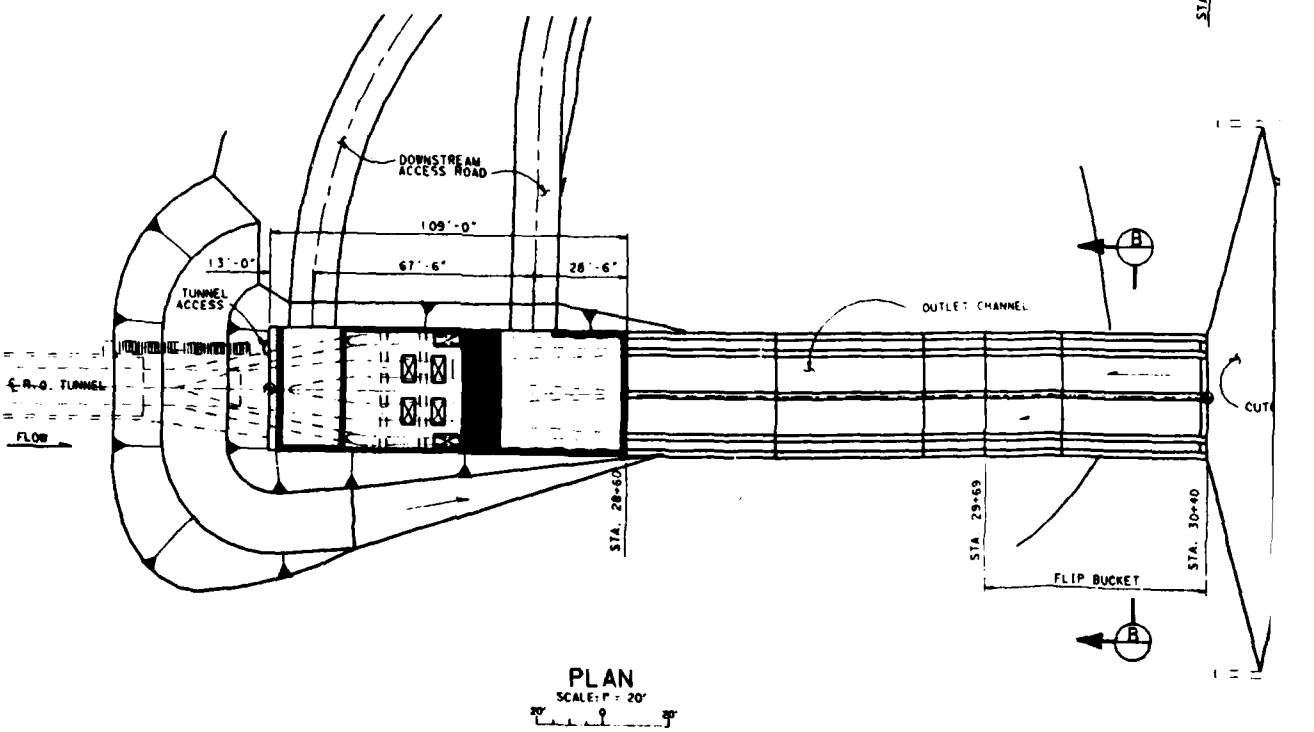
1. SEE APPENDIX "B" FOR MORE OUTLET WORKS DETAILS.

U.S. ARMY ENGINEER DISTRICT PORTLAND CORPS OF ENGINEERS	U.S. ARMY ENGINEER DISTRICT LOS ANGELES CORPS OF ENGINEERS
DESIGNED BY: B. CHAMBERS	SANTA ANA RIVER MAINSTEM, CALIFORNIA PHASE II GENERAL DESIGN MEMORANDUM
DRAWN BY: S. ERICSON	SEVEN OAKS DAM OUTLET WORKS
CHECKED BY: K. SWANSON	DOWNSTREAM CONTROL
SUBMITTED BY: P. E. DAUGHERTY, P.E.	DIVERSION/OUTLET TUNNEL PLAN, PROFILE & SECTIONS
DATE APPROVED:	SPEC. NO. 84CW0- -0-
	DISTRICT FILE NO. OAK-20-3/2
	PLATE 9

CORPS OF ENGINEERS

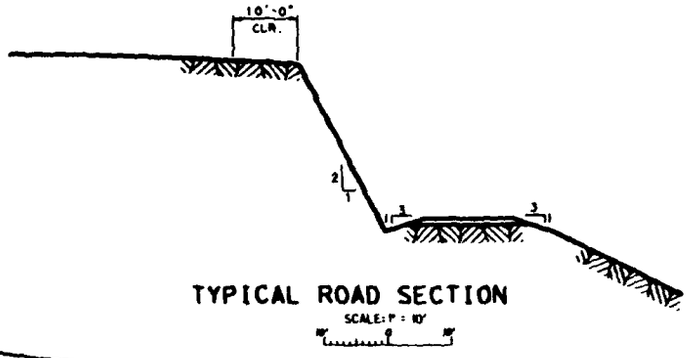


PROFILE
SCALE: 1" = 20'



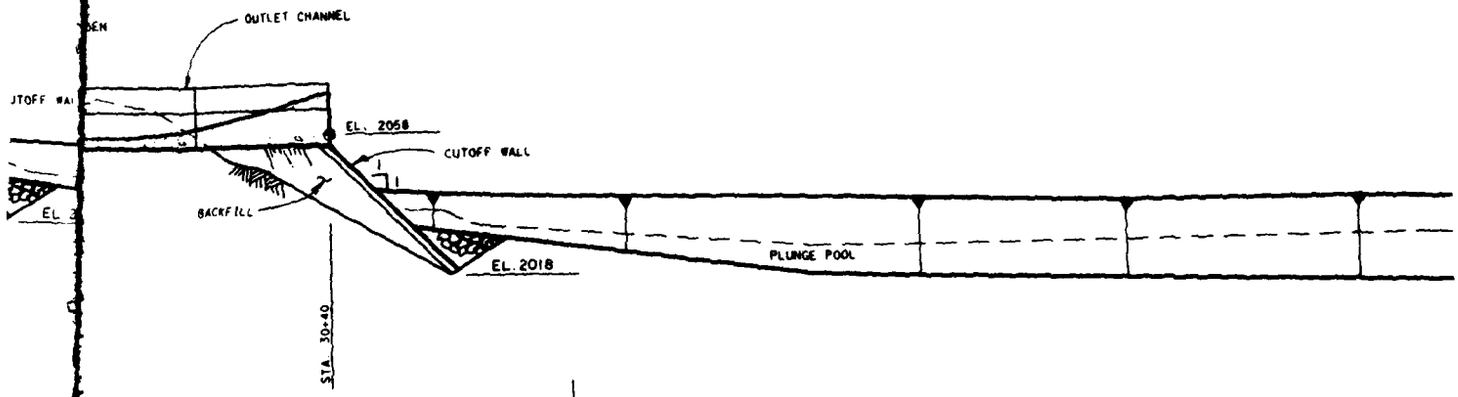
PLAN
SCALE: 1" = 20'

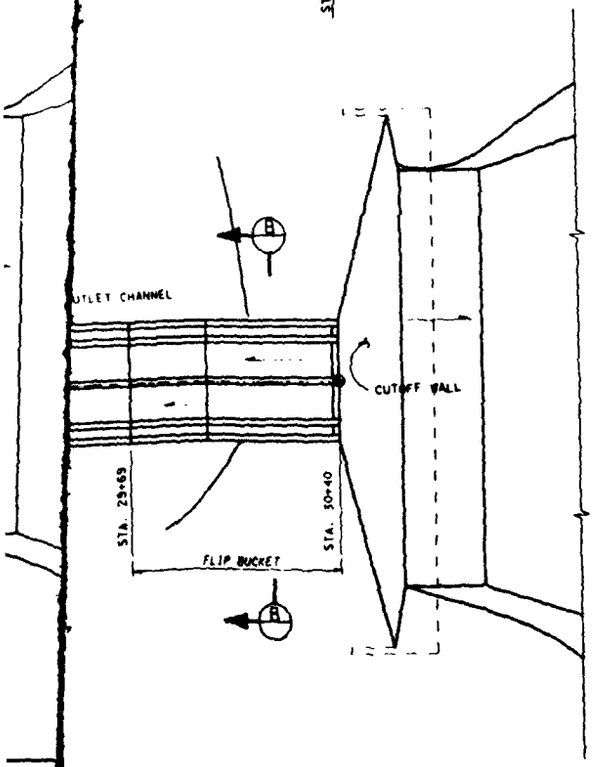
U.S. ARMY



TYPICAL ROAD SECTION

SCALE: 1" = 10'

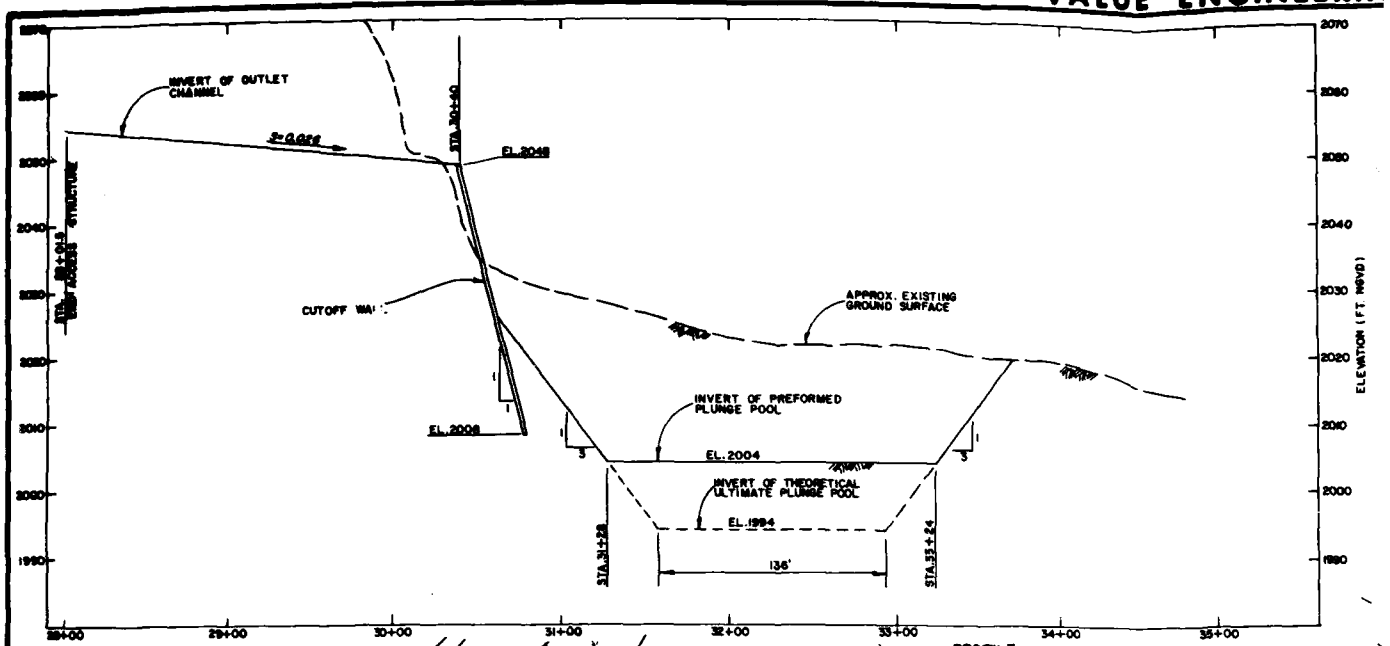




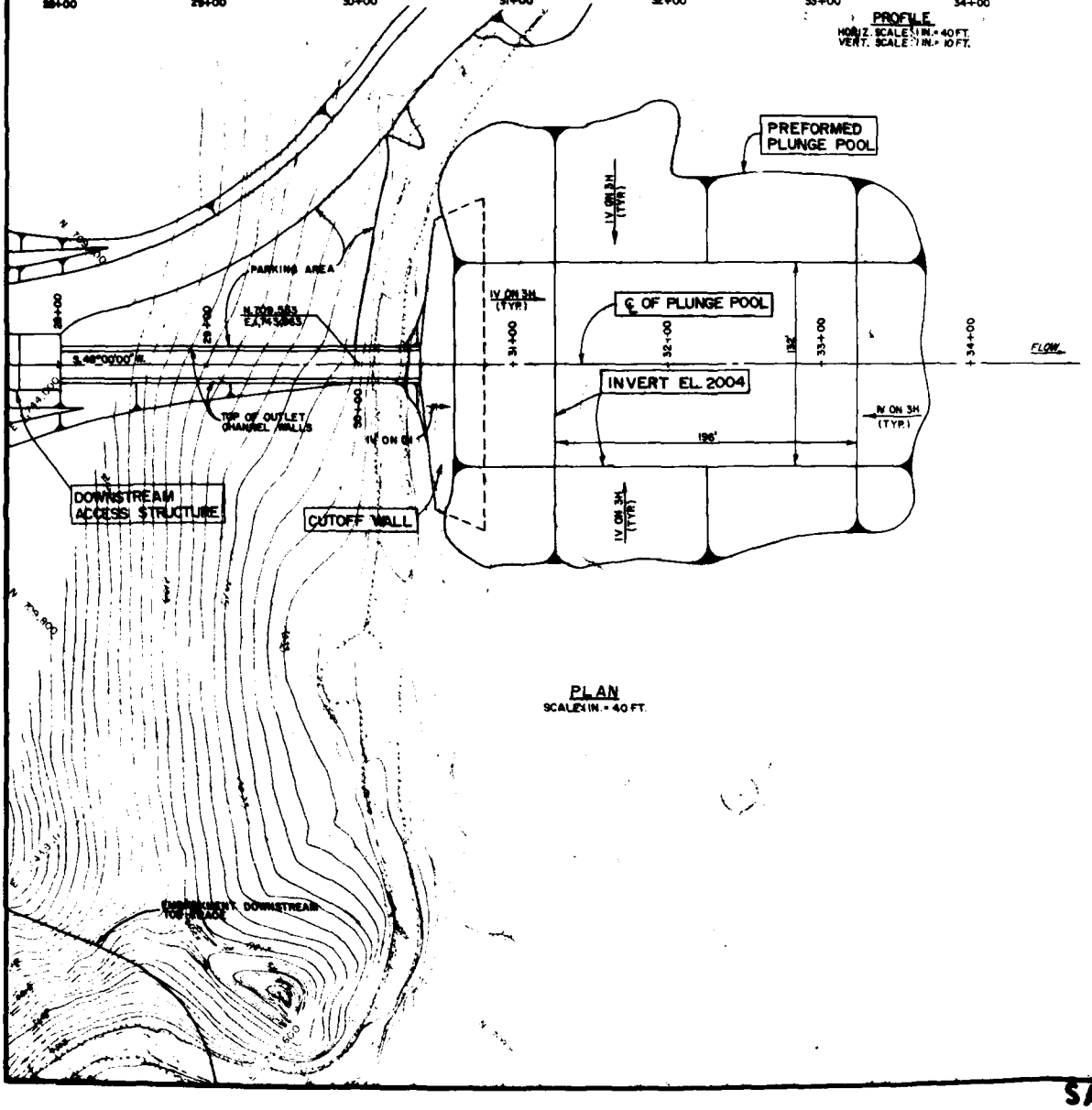
NOTE:

1. SEE APPENDIX "B" FOR MORE OUTLET WORKS DETAILS.

U.S. ARMY ENGINEER DISTRICT PORTLAND CORPS OF ENGINEERS	U.S. ARMY ENGINEER DISTRICT LOS ANGELES CORPS OF ENGINEERS
DESIGNED BY: B. CHAMBERS	SANTA ANA RIVER MAINSTEM, SAN TIMOTEO CANYON PHASE II GENERAL DESIGN MEMORANDUM
DRAWN BY: S. ERICSON	SEVEN OAKS DAM OUTLET WORKS DOWNSTREAM CONTROL DOWNSTREAM STRUCTURES
CHECKED BY: E. TOMSON	
SUBMITTED BY: W. S. DOUGHERTY, P.E.	DATE APPROVED:
	SPEC. NO.
CADD GENERATED DESIGN FILE:	DISTRICT FILE NO. OAK-211-3/5
	PLATE 10

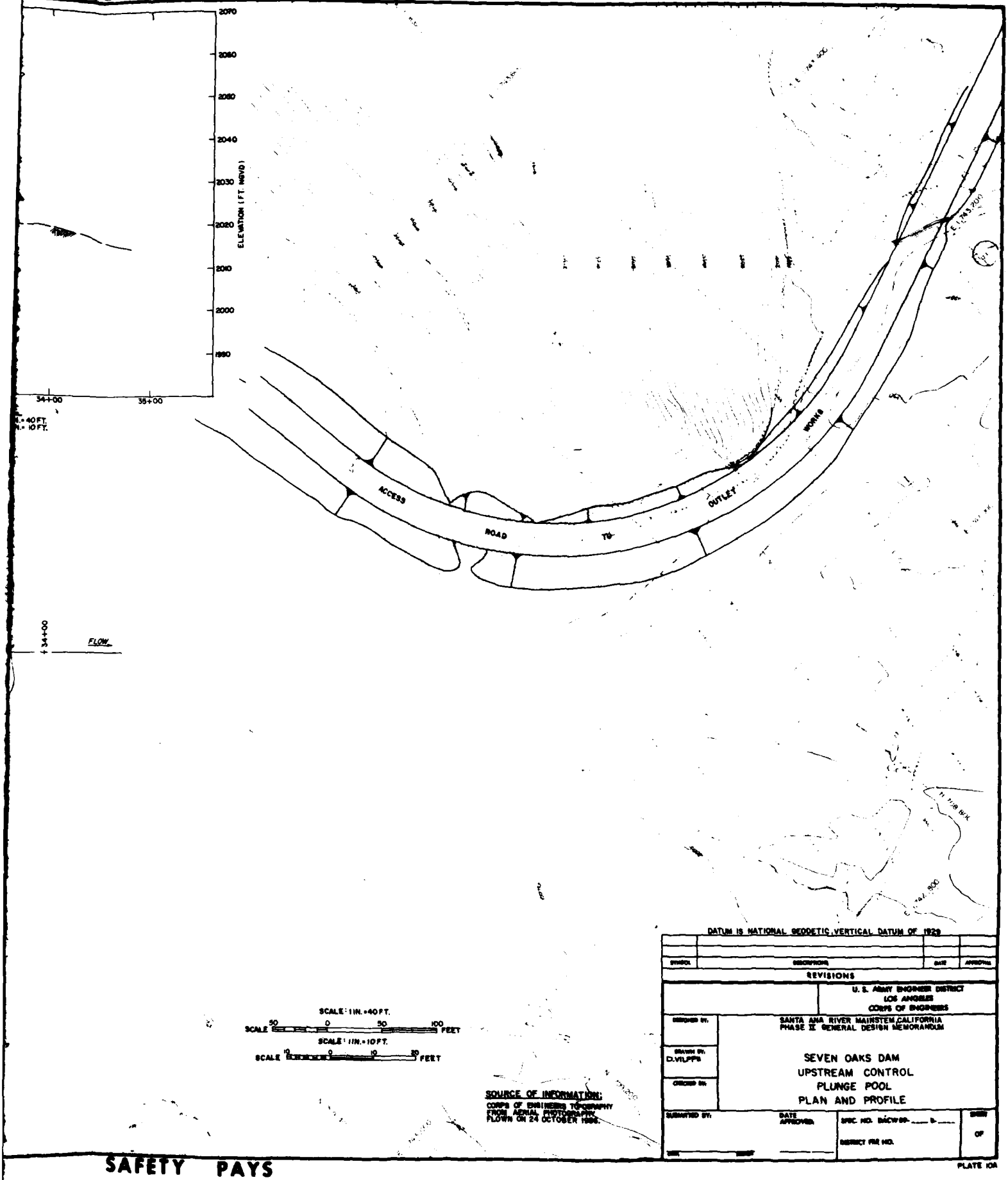


PROFILE
HORIZ. SCALE: 1 IN. = 40 FT.
VERT. SCALE: 1 IN. = 10 FT.



PLAN
SCALE: 1 IN. = 40 FT.

VALUE ENGINEERING PAYS



SOURCE OF INFORMATION:
CORPS OF ENGINEERS TOPOGRAPHY FROM AERIAL PHOTOGRAPHY FLOWN ON 24 OCTOBER 1986.

DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

SYMBOL	DESCRIPTION	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY: D.VILPPS
CHECKED BY:

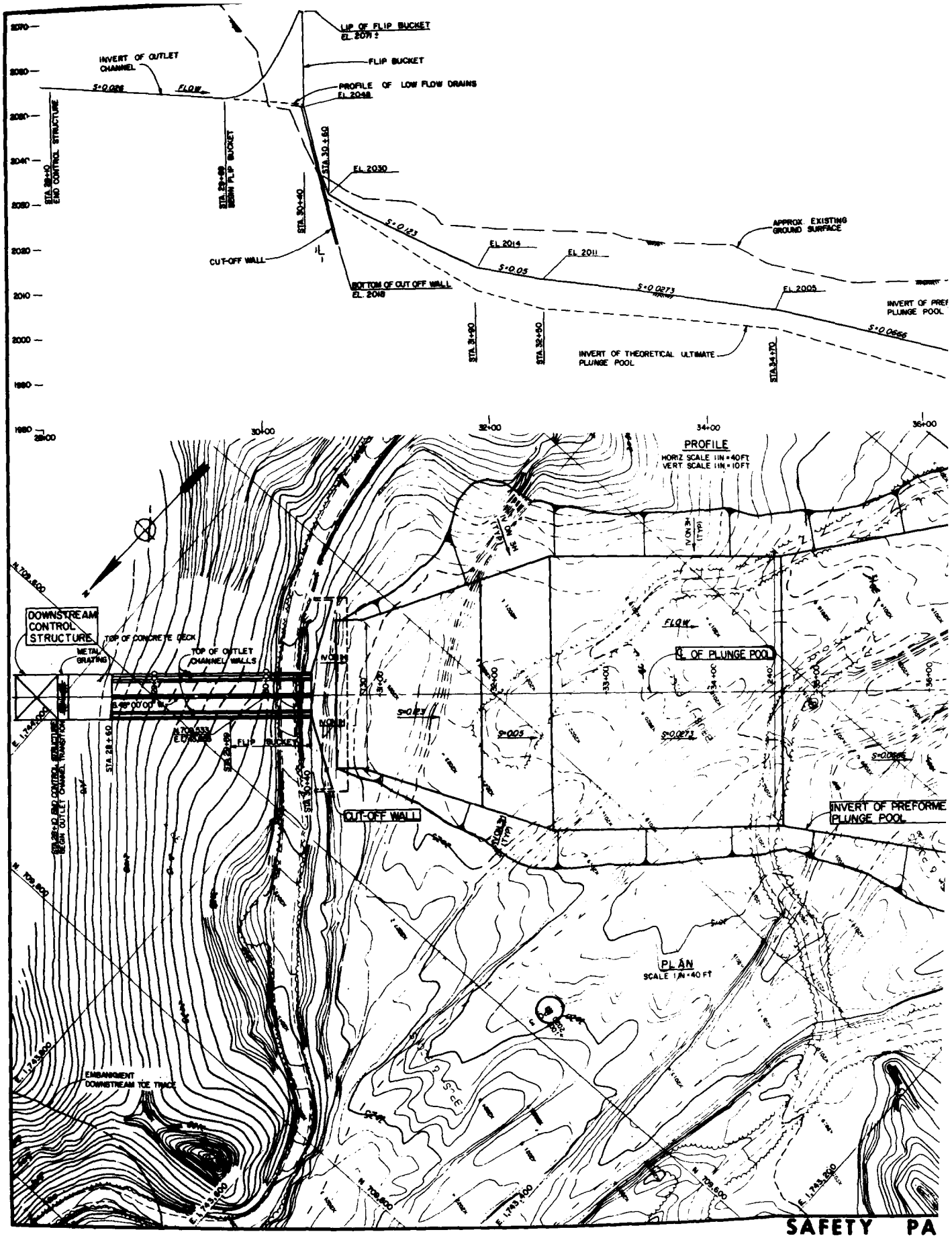
SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM
UPSTREAM CONTROL
PLUNGE POOL
PLAN AND PROFILE

SUBMITTED BY: DATE APPROVED:
SPEC. NO. DACW09-___-___
DISTRICT FILE NO.
SHEET OF

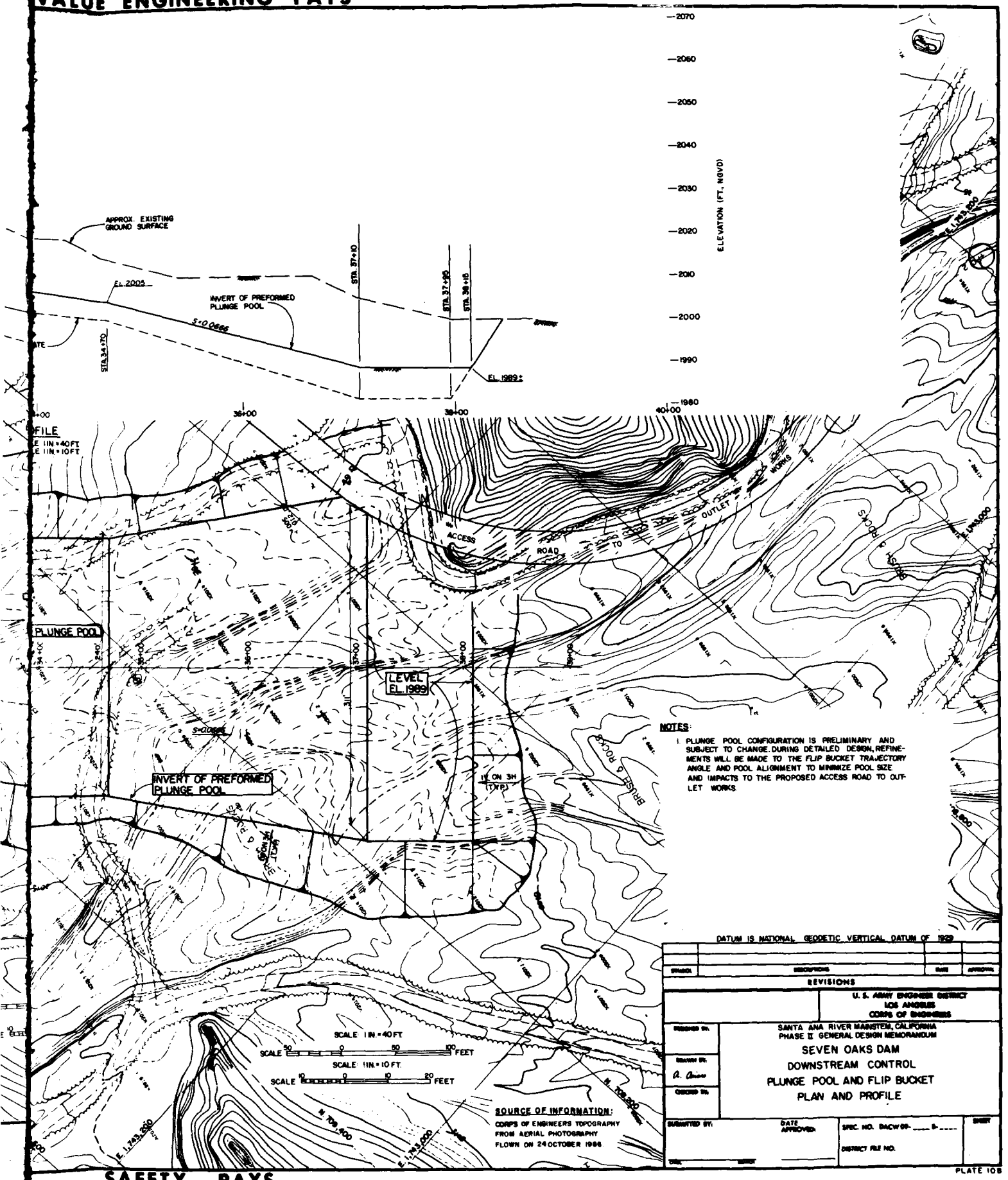
PLATE 10A

SAFETY PAYS


LIP OF FLIP BUCKET EL. 2011 ±
FLIP BUCKET
PROFILE OF LOW FLOW DRAINS EL. 2048
INVERT OF OUTLET CHANNEL
S=0.026
FLOW
STA 30+60
EL. 2030
STA 30+40
STA 28+0 END CONTROL STRUCTURE
STA 29+69 BEGIN FLIP BUCKET
CUT-OFF WALL
BOTTOM OF CUT OFF WALL EL. 2048
S=0.123
EL. 2014
EL. 2011
S=0.05
S=0.0273
EL. 2005
APPROX. EXISTING GROUND SURFACE
INVERT OF THEORETICAL ULTIMATE PLUNGE POOL
STA 31+90
STA 32+90
STA 34+70
INVERT OF PREF PLUNGE POOL
S=0.0666
2070
2060
2050
2040
2030
2020
2010
2000
1990
1980
28+00
30+00
32+00
34+00
36+00
PROFILE
HORIZ SCALE 1 IN = 40 FT
VERT SCALE 1 IN = 10 FT
DOWNSTREAM CONTROL STRUCTURE
TOP OF CONCRETE DECK
METAL GRATING
TOP OF OUTLET CHANNEL WALLS
N 709,800
E 1744,000
E 1743,800
E 1743,600
E 1743,400
E 1743,200
N 709,600
N 709,400
N 709,200
FLOW
℄ OF PLUNGE POOL
CUT-OFF WALL
INVERT OF PREFORME PLUNGE POOL
EMBANKMENT DOWNSTREAM TOE TRACE
PLAN
SCALE 1 IN = 40 FT
ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

SAFETY PA

VALUE ENGINEERING PAYS



NOTES:

1. PLUNGE POOL CONFIGURATION IS PRELIMINARY AND SUBJECT TO CHANGE. DURING DETAILED DESIGN, REFINEMENTS WILL BE MADE TO THE FLIP BUCKET TRAJECTORY ANGLE AND POOL ALIGNMENT TO MINIMIZE POOL SIZE AND IMPACTS TO THE PROPOSED ACCESS ROAD TO OUTLET WORKS.

SOURCE OF INFORMATION:
CORPS OF ENGINEERS TOPOGRAPHY FROM AERIAL PHOTOGRAPHY FLOWN ON 24 OCTOBER 1986.

DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY: R. Chinn
CHECKED BY:

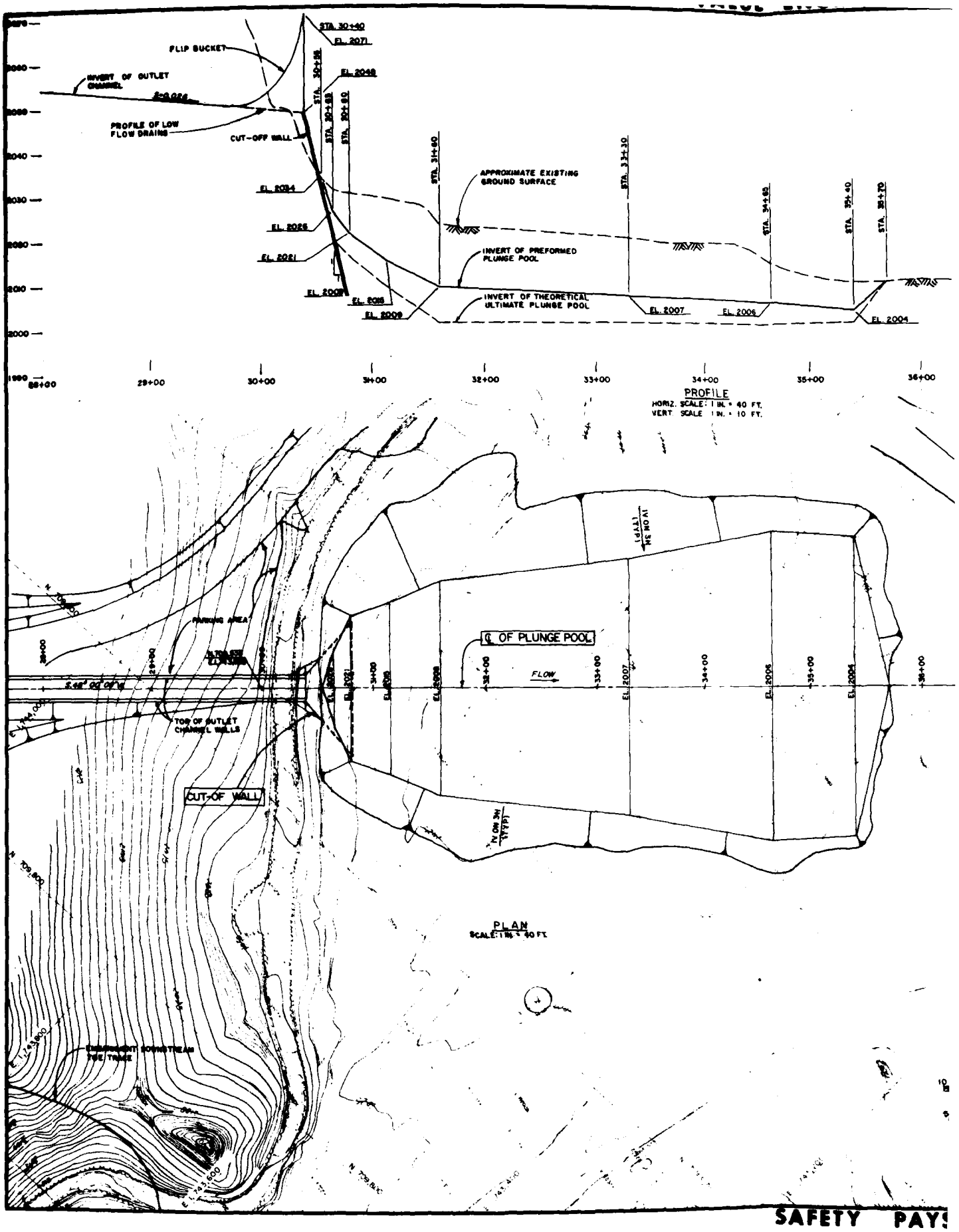
SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM
DOWNSTREAM CONTROL
PLUNGE POOL AND FLIP BUCKET
PLAN AND PROFILE

SUBMITTED BY: DATE APPROVED:
SPEC. NO. SACW 09-____-D-____
DISTRICT FILE NO.
SHEET

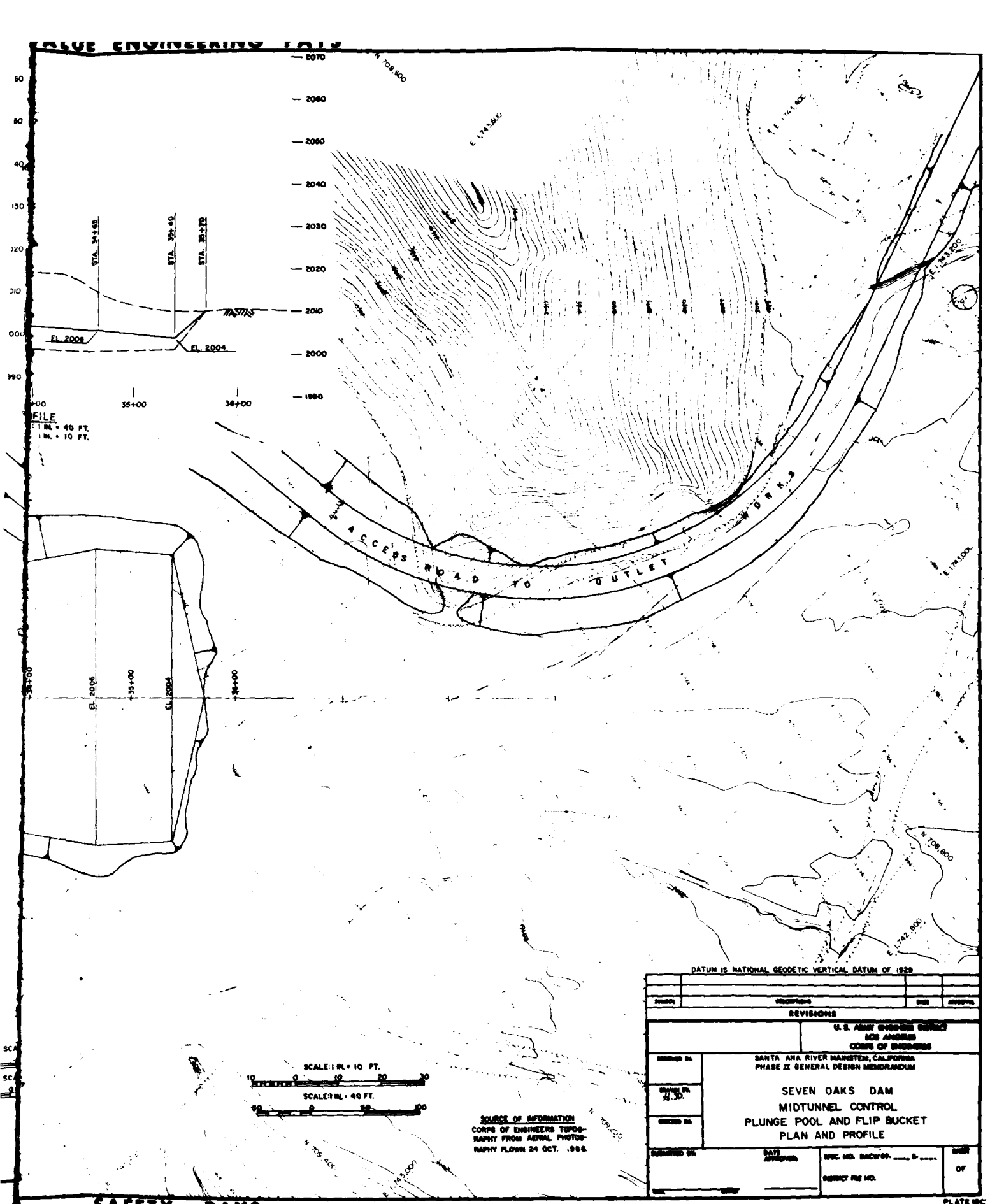
PLATE 108

SAFETY PAYS


FLIP BUCKET
INVERT OF OUTLET CHANNEL
PROFILE OF LOW FLOW DRAINS
CUT-OFF WALL
STA. 30+40
EL. 2071
EL. 2048
EL. 2034
EL. 2026
EL. 2021
EL. 2009
EL. 2008
APPROXIMATE EXISTING GROUND SURFACE
INVERT OF PREFORMED PLUNGE POOL
INVERT OF THEORETICAL ULTIMATE PLUNGE POOL
EL. 2007
EL. 2006
EL. 2004
PROFILE
HORIZ. SCALE: 1 IN. = 40 FT.
VERT. SCALE 1 IN. = 10 FT.
PARKING AREA
TOP OF OUTLET CHANNEL WALLS
CUT-OF WALL
℄ OF PLUNGE POOL
FLOW
EMBANKMENT DOWNSTREAM TOE TRACE
PLAN
SCALE: 1 IN. = 40 FT.

SAFETY PAY

VALUE ENGINEERING PAYS



DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

DESIGNED BY	DESCRIPTION	DATE	APPROVAL
	REVISIONS		

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY

DRAWN BY
M. D.

CHECKED BY

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM
MIDTUNNEL CONTROL
PLUNGE POOL AND FLIP BUCKET
PLAN AND PROFILE

SUBMITTED BY

DATE APPROVED

SPEC. NO. DACW09-___-___

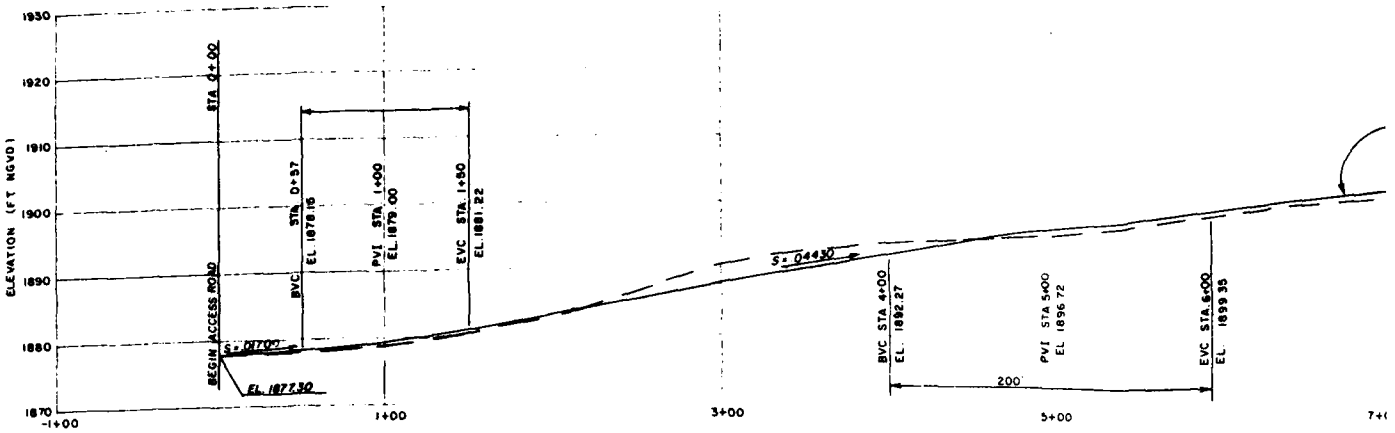
DISTRICT FILE NO.

SHEET OF

SOURCE OF INFORMATION
CORPS OF ENGINEERS TOPOGRAPHY FROM AERIAL PHOTOGRAPHY FLOWN 24 OCT. 1986.

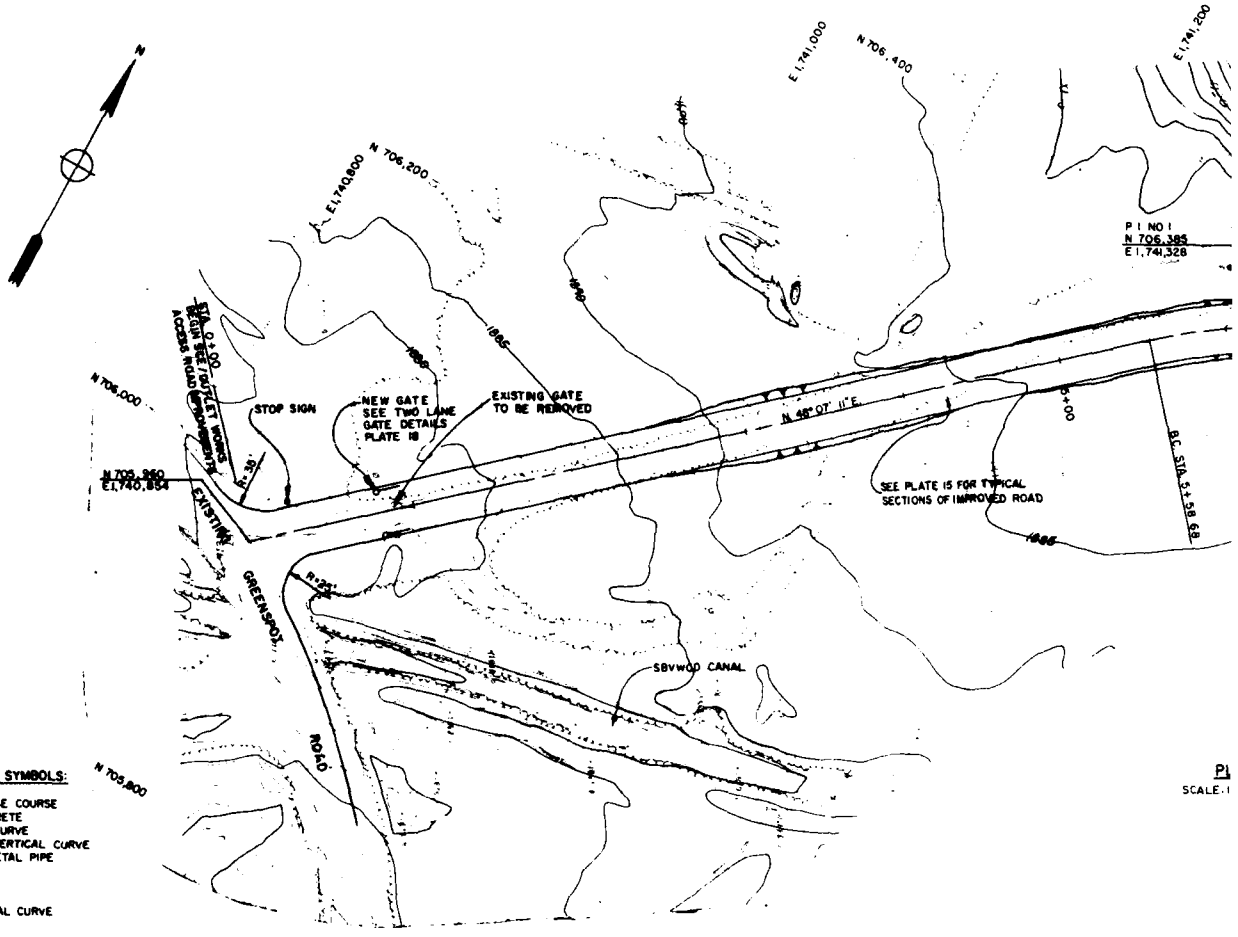
PLATE 10C

SAFETY PAYS



PROFILE

HORIZ. SCALE: 1 IN = 40 FT
VERT SCALE: 1 IN = 10 FT



ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

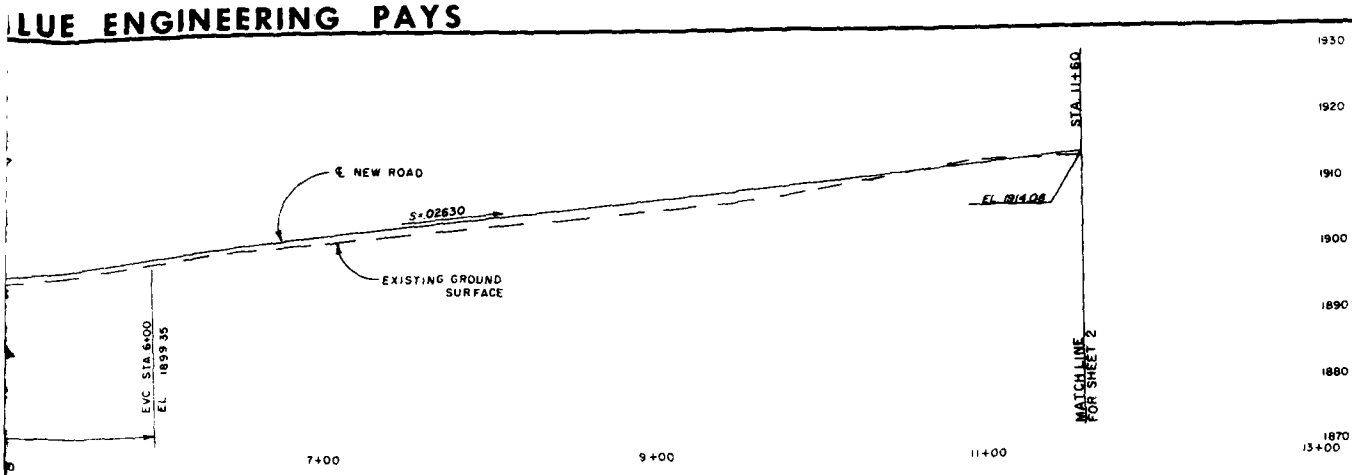
ABBREVIATIONS AND SYMBOLS:

A.B.C.	AGGREGATE BASE COURSE
A.C.	ASPHALT CONCRETE
B.C.	BEGINNING OF CURVE
B.V.C.	BEGINNING OF VERTICAL CURVE
C.M.P.	CORRUGATED METAL PIPE
DIA.	DIAMETER
E.C.	END OF CURVE
EL.	ELEVATION
E.V.C.	END OF VERTICAL CURVE
FT.	FEET
HORIZ.	HORIZONTAL
IN.	INCHES
L.	LENGTH
NO.	NUMBER
P.I.	POINT OF INTERSECTION
P.V.I.	POINT OF VERTICAL INTERSECTION
R.	RADIUS
S.	SLOPE
SBVWCD	SAN BERNARDINO VALLEY WATER CONSERVATION DISTRICT
S.C.E.	SOUTHERN CALIFORNIA EDISON
STA.	STATION
T.	TANGENT
T.S.	TUBE STEEL
VERT.	VERTICAL
W.W.F.	WELDED WIRE FABRIC
Δ	DEFLECTION ANGLE
℄	CENTERLINE
— · —	CENTERLINE
	TELEPHONE POLE
	C.M.P.
	GATE

PI
SCALE:1

SAFETY PA

LUE ENGINEERING PAYS



PROFILE
HORIZ SCALE 1 IN = 40 FT
VERT SCALE 1 IN = 10 FT

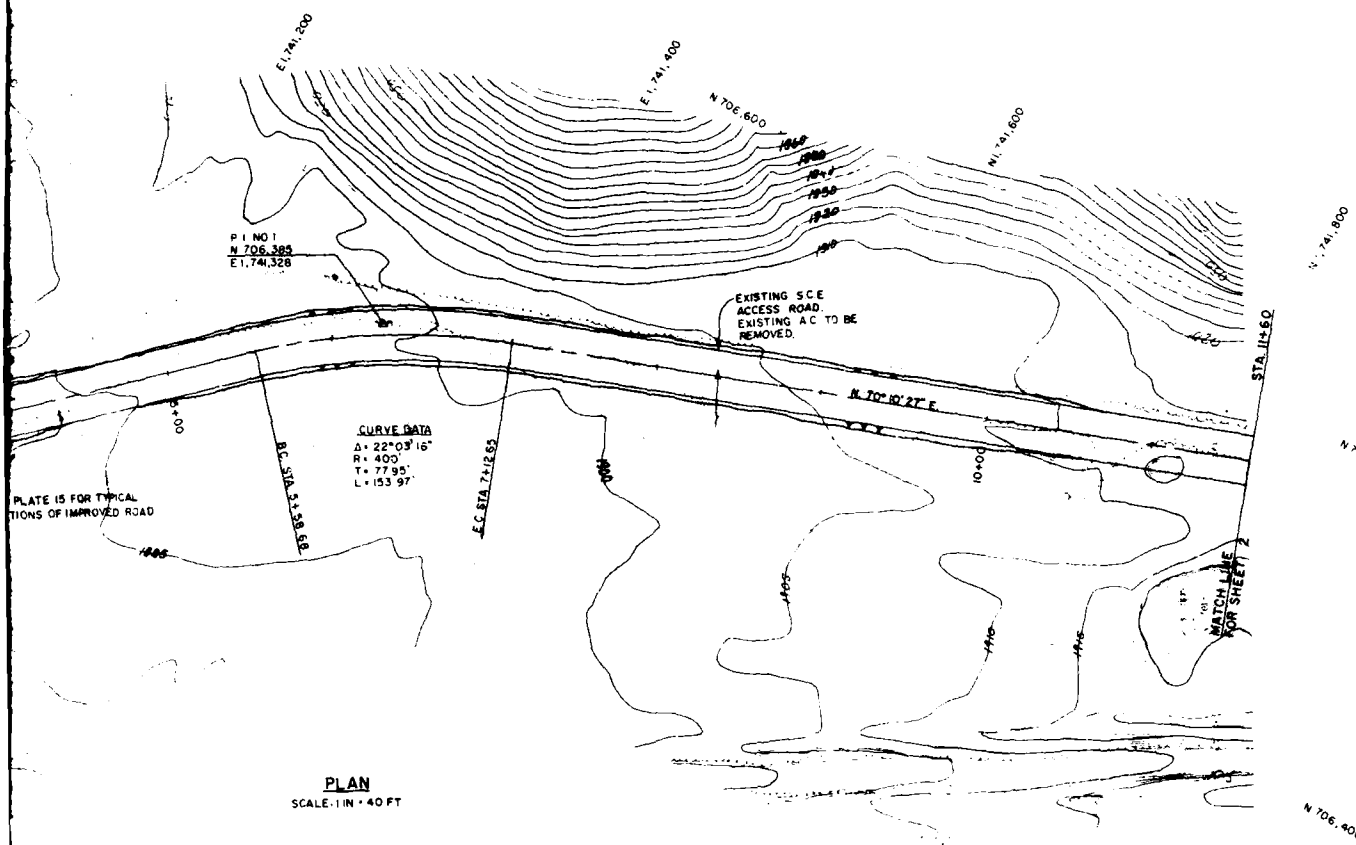


PLATE 15 FOR TYPICAL TIONS OF IMPROVED ROAD

PLAN
SCALE 1 IN = 40 FT

SCALE 1 IN = 10 FT
10 0 10 20 30
SCALE 1 IN = 40 FT
50 0 50 100

DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY
DRAWN BY
CHECKED BY
SUBMITTED BY
DATE APPROVED
CHIEF, DIVISION

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

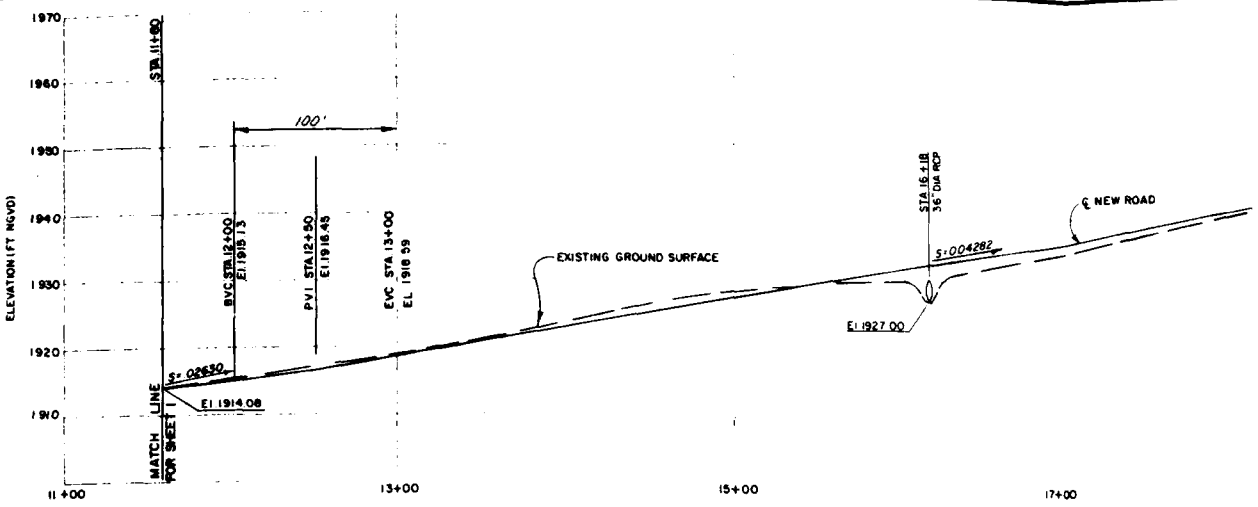
OUTLET WORKS ACCESS ROAD
PLAN AND PROFILE
STA 0+00 TO STA 11+60

SPEC. NO. DACW09-___-___
DISTRICT FILE NO.

SHEET 1 OF 5

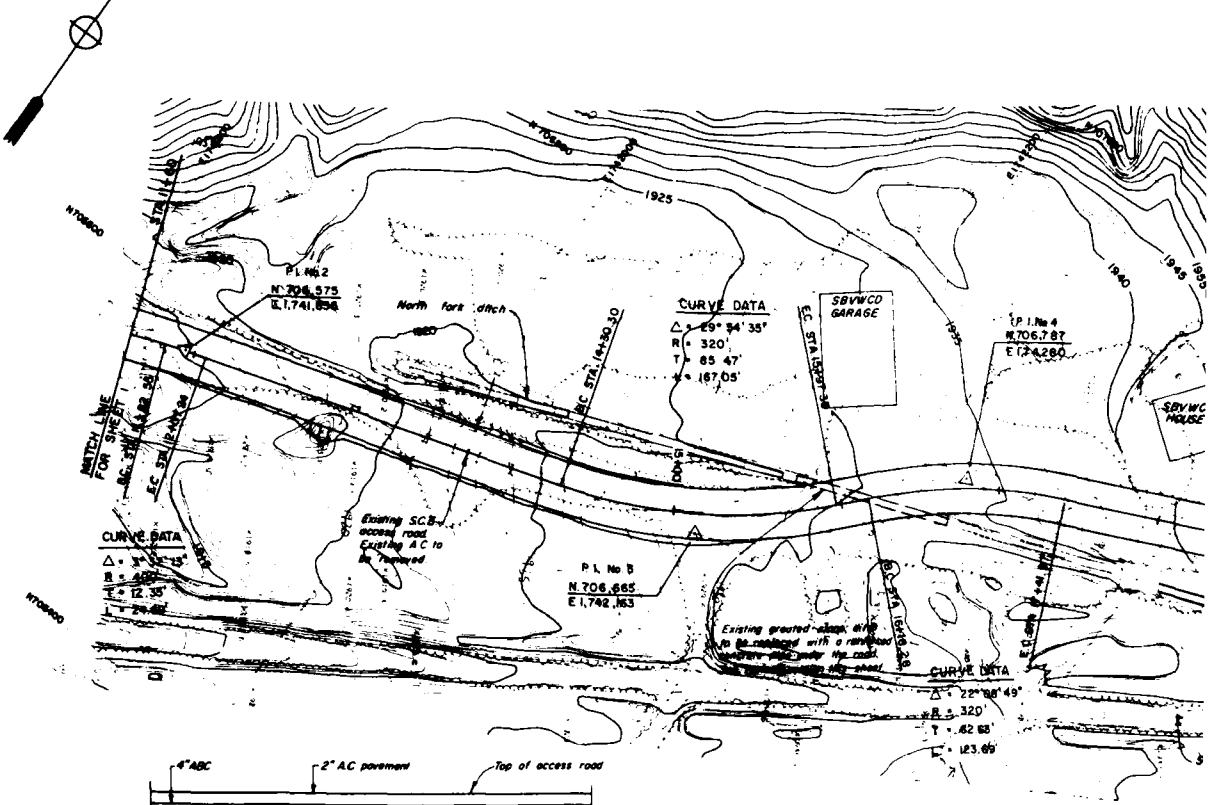
PLATE 11

SAFETY PAYS



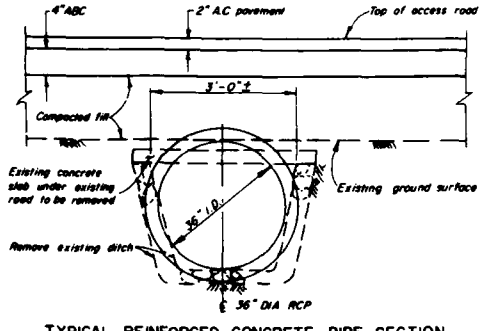
PROFILE

HORIZ SCALE 1 IN.
VERT SCALE 1 IN.



PLAN

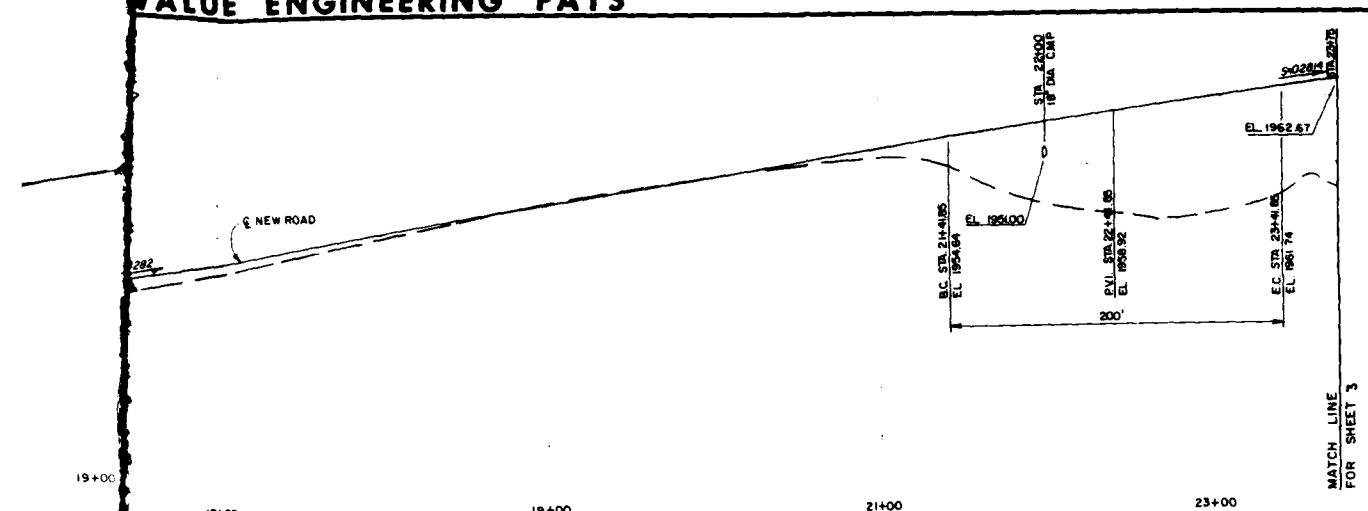
SCALE 1 IN.



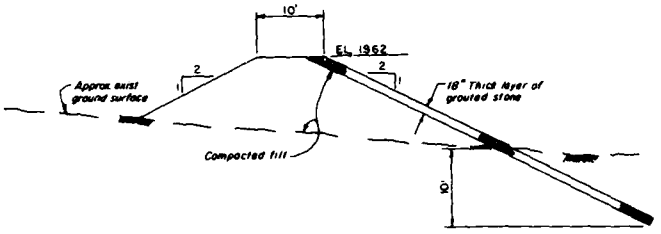
TYPICAL REINFORCED CONCRETE PIPE SECTION
NOT TO SCALE

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

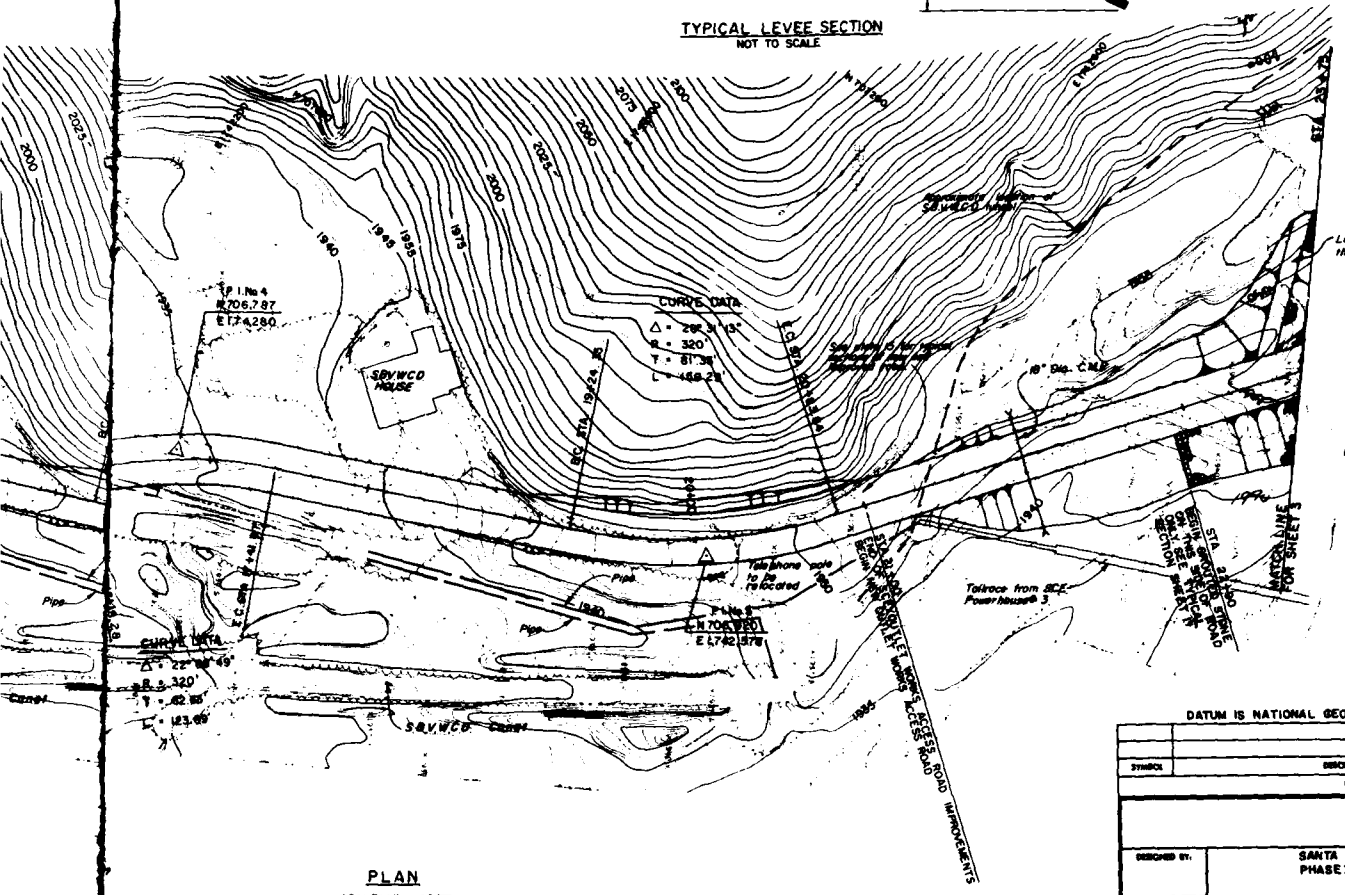
IG PA
VALUE ENGINEERING PAYS



PROFILE
HORIZ. SCALE: 1 IN. = 40 FT.
VERT. SCALE: 1 IN. = 10 FT.



TYPICAL LEVEE SECTION
NOT TO SCALE



PLAN
SCALE: 1 IN. = 40 FT.

SCALE: 1 IN. = 10 FT.
10 0 10 20 30

SCALE 1 IN. = 40 FT.
50 0 50 100

NOTES:
1. GROUTED STONE MAY BE REPLACED WITH OVERSIZED STONE (+9" TO 4' DIA.) FROM THE STREAMBED.
2. EXISTING UTILITIES TO BE AVOIDED OR PROTECTED IN PLACE EXCEPT AS NOTED.

DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

SYMBOL	DESCRIPTIONS	DATE	APPD
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY: F. Castellanos
CHECKED BY:

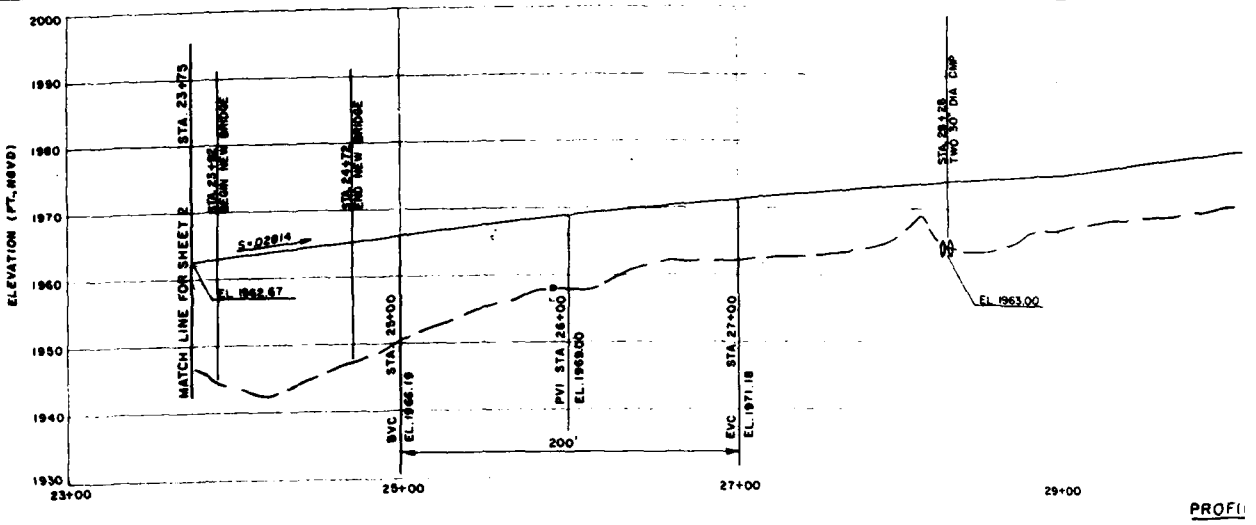
SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

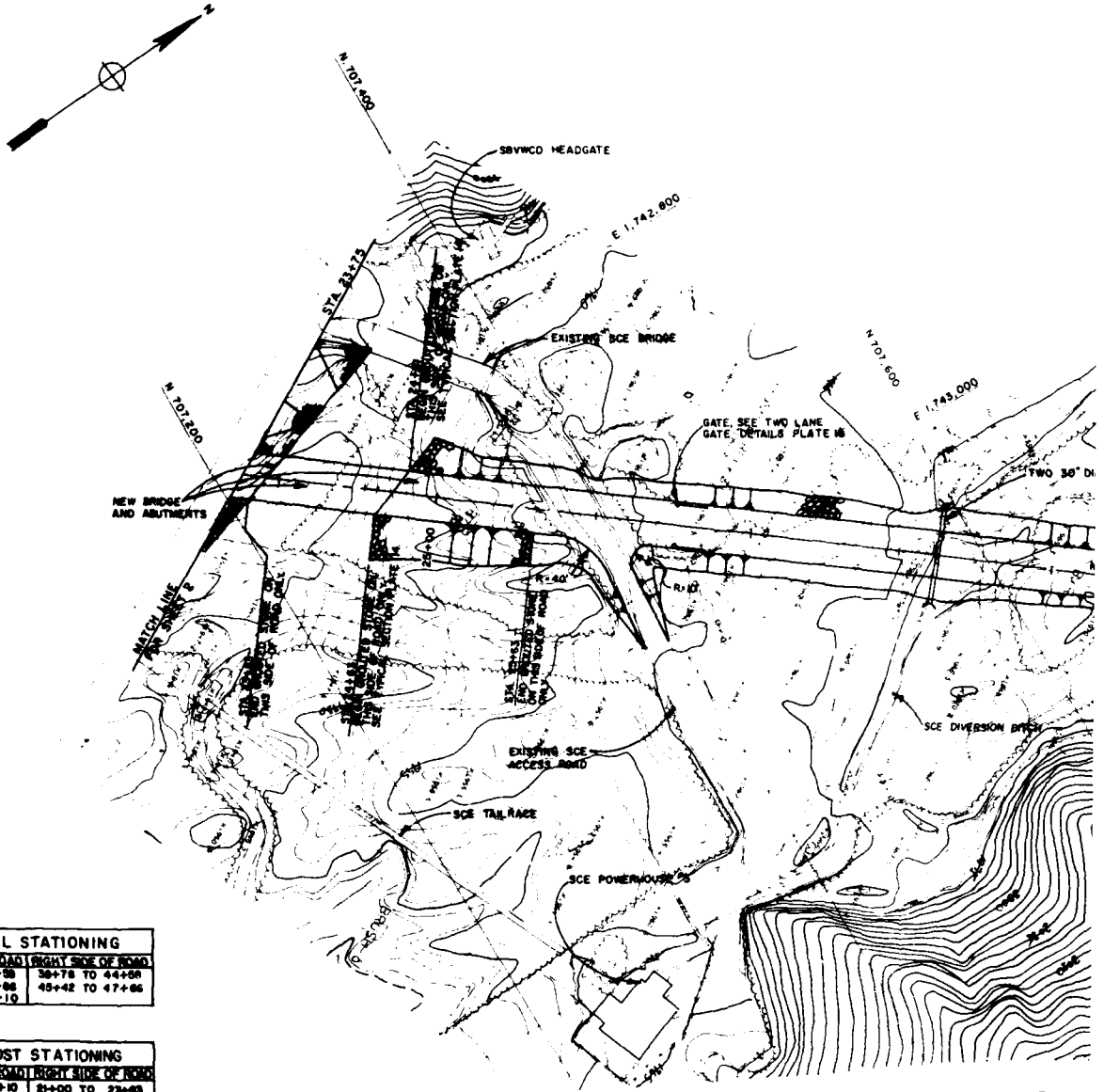
OUTLET WORKS ACCESS ROAD
PLAN, PROFILE AND SECTIONS
STA. 11+60 TO STA. 23+75

SUBMITTED BY: | DATE APPROVED | SPEC. NO. DACW09- ---- 5- ---- | DISTRICT FILE NO.

'S
SAFETY PAYS



PROFI
HORIZ SCALE
VERT SCALE



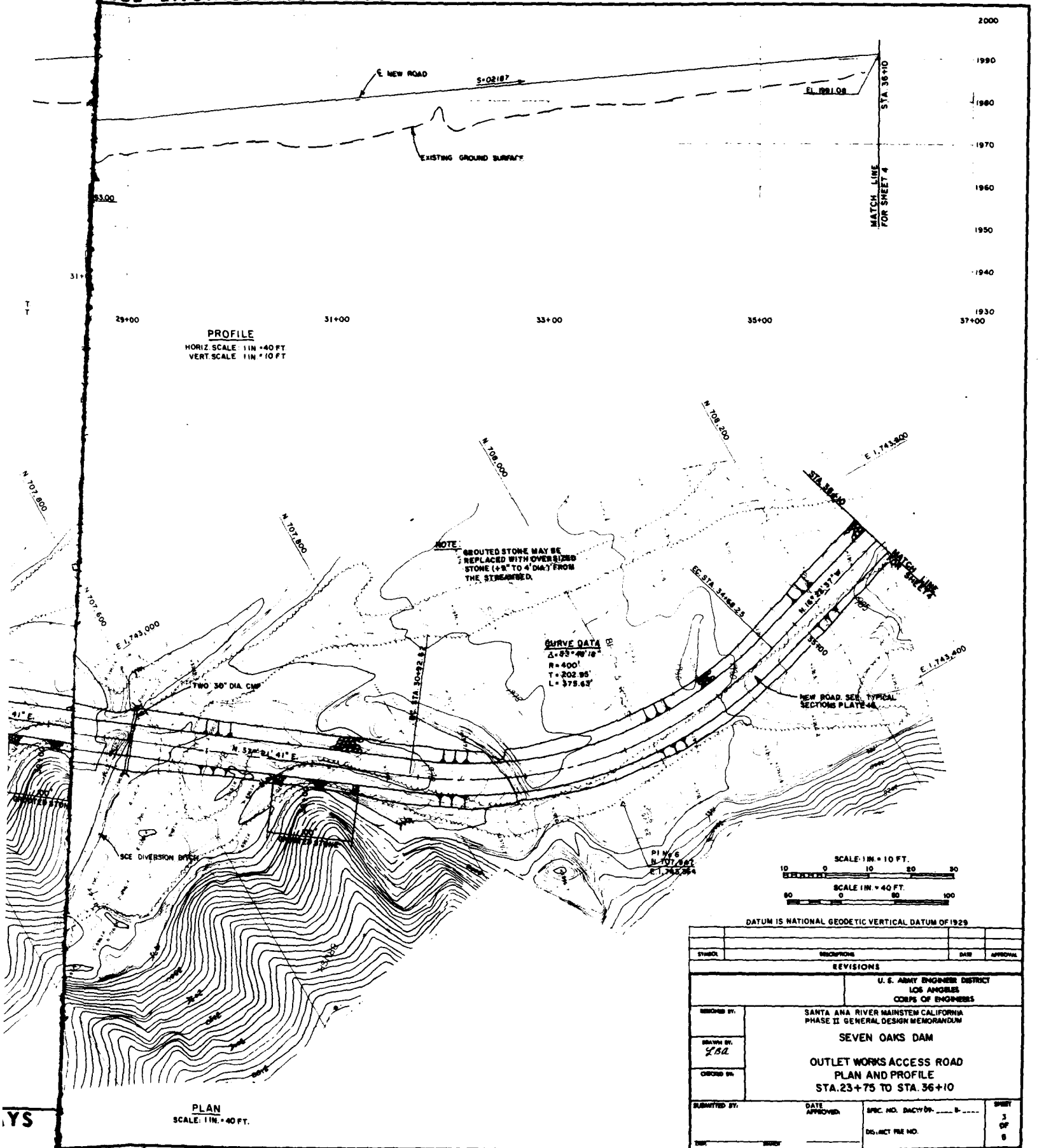
PLAN
SCALE: 1 IN.=

GUARDRAIL STATIONING	
LEFT SIDE OF ROAD	RIGHT SIDE OF ROAD
38+78 TO 44+58	38+78 TO 44+58
45+42 TO 47+86	45+42 TO 47+86
48+98 TO 52+10	

GUIDEPOST STATIONING	
LEFT SIDE OF ROAD	RIGHT SIDE OF ROAD
21+00 TO 24+10	21+00 TO 23+63
24+81 TO 27+00	24+63 TO 27+00
30+92 TO 34+58	30+92 TO 34+58
47+86 TO 48+98	44+58 TO 48+42

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

IG P
LUE ENGINEERING PAYS



DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

SYMBOL	DESCRIPTION	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY: LBA
CHECKED BY:

SANTA ANA RIVER MAINSTEM CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

OUTLET WORKS ACCESS ROAD
PLAN AND PROFILE
STA. 23+75 TO STA. 36+10

SUBMITTED BY: DATE APPROVED:

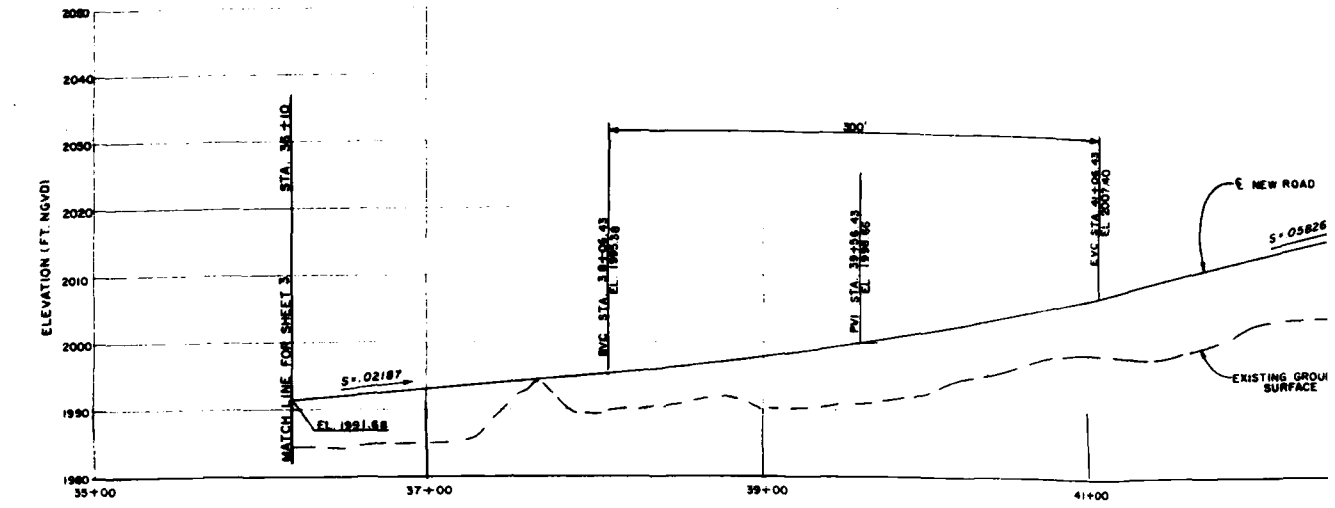
SPEC. NO. DACW09-__-__-__

DISTRICT FILE NO.

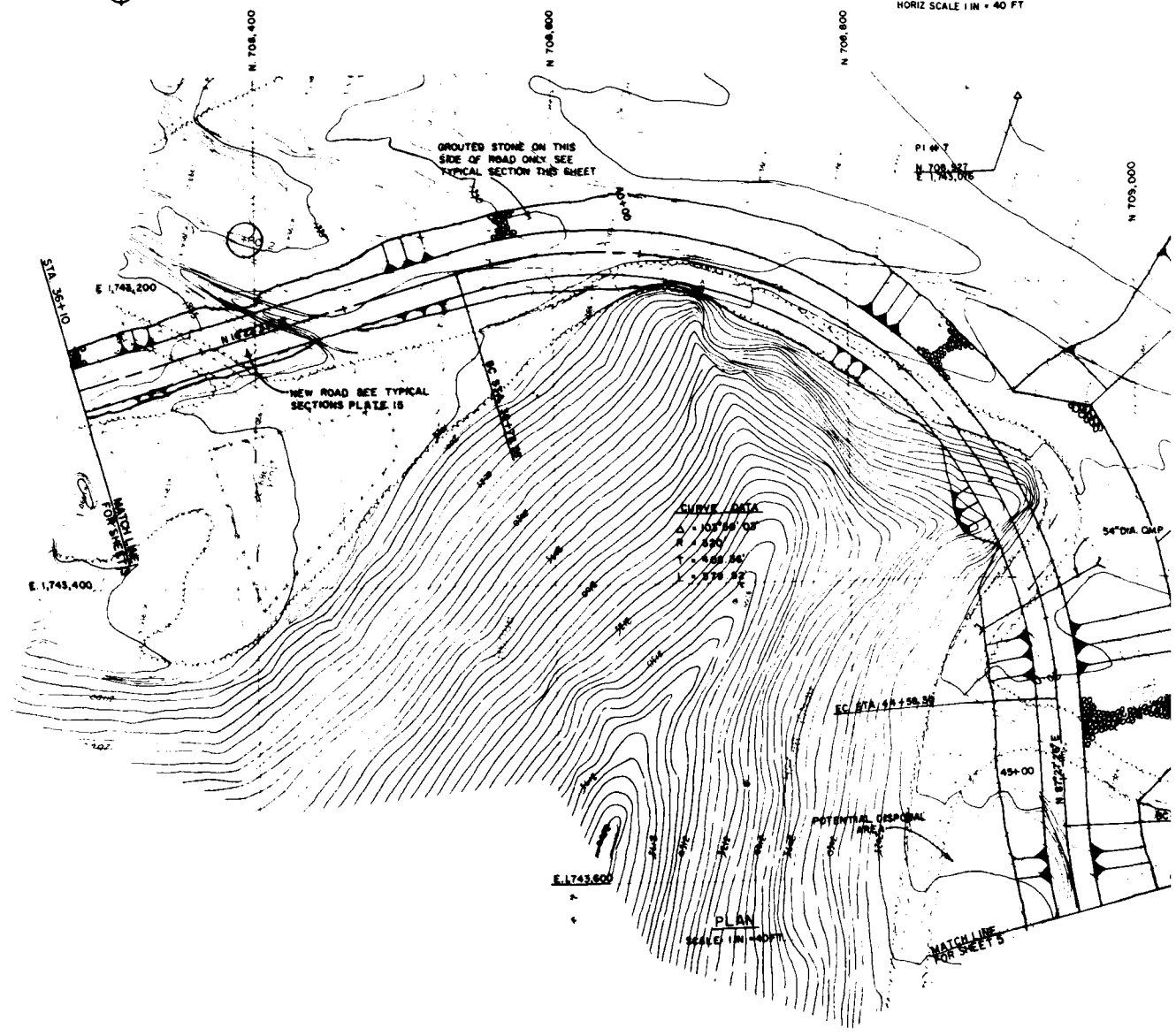
SHEET 3 OF 8

PLATE 18

YS
SAFETY PAYS



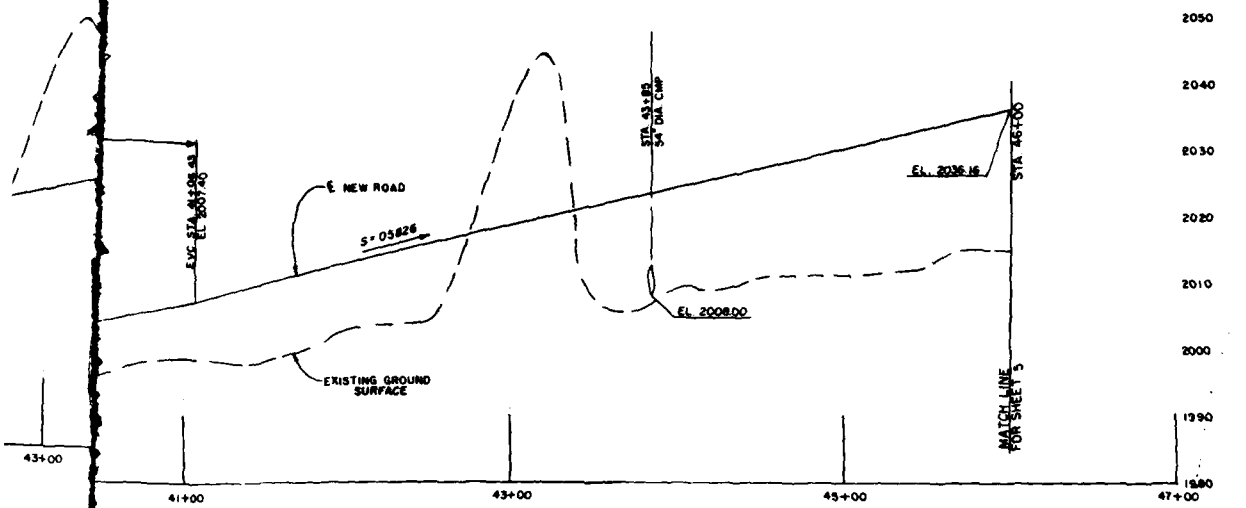
PROFILE
VERT SCALE 1 IN = 10 FT
HORIZ SCALE 1 IN = 40 FT



ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

PA

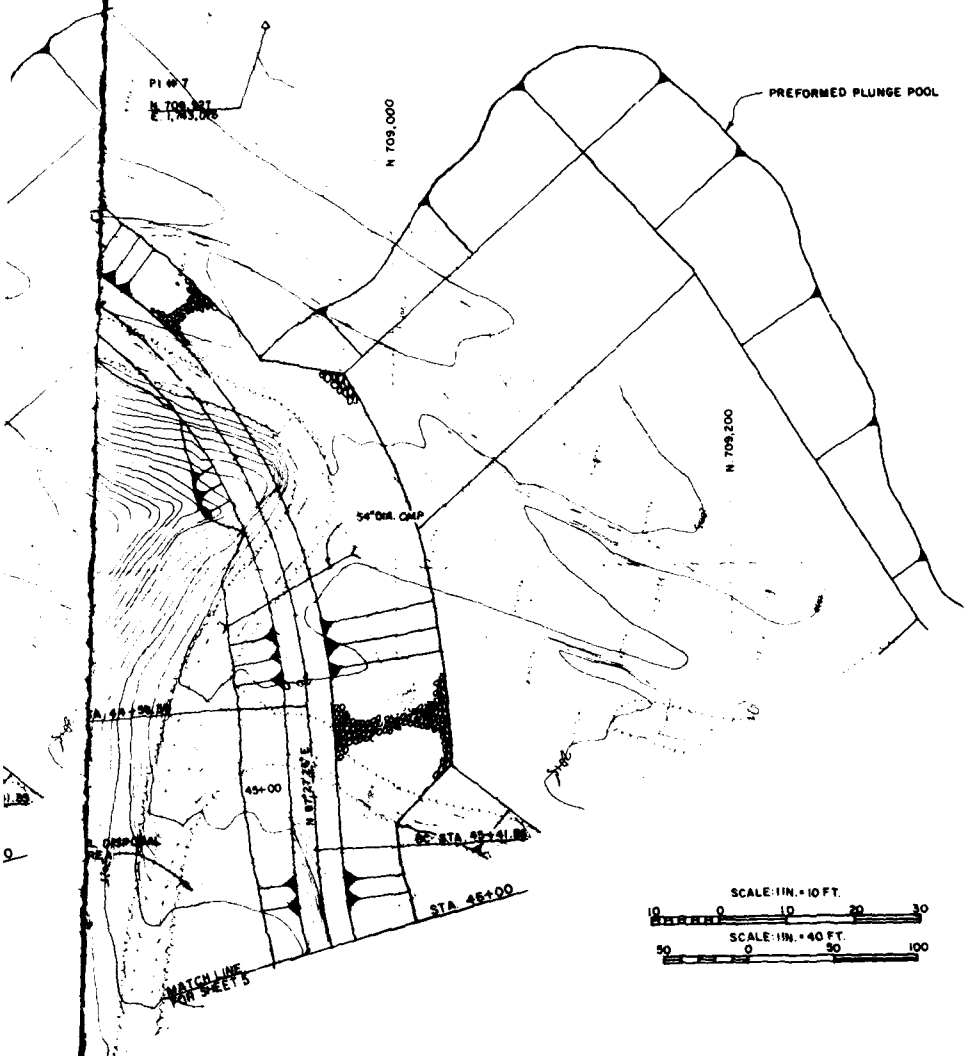
ALUE ENGINEERING PAYS



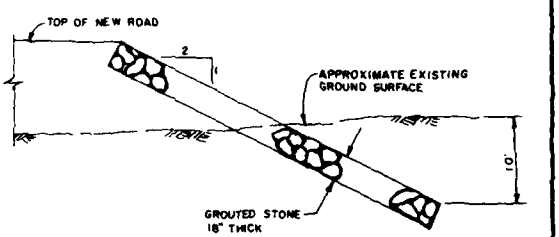
PROFILE

VERT SCALE: 1 IN = 10 FT

HORIZ SCALE 1 IN = 40 FT



NOTE:
GROUTED STONE MAY BE REPLACED WITH OVERSIZED STONE (+9" TO 4' DIA.) FROM THE STREAMBED.



TYPICAL GROUTED STONE SECTION
NOT TO SCALE

SCALE: 1 IN. = 10 FT.
10 0 10 20 30
SCALE: 1 IN. = 40 FT.
50 0 50 100

DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

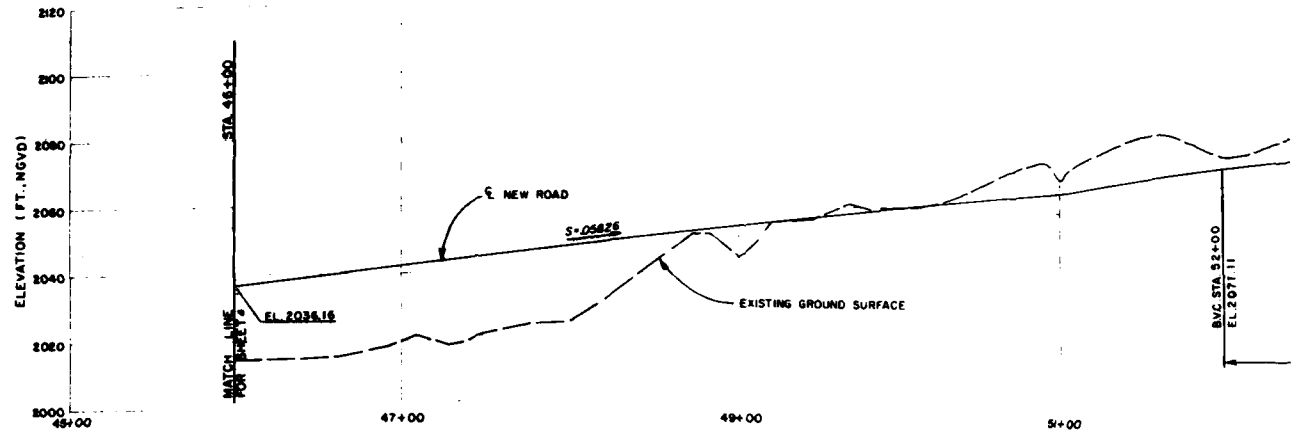
SEVEN OAKS DAM

OUTLET WORKS ACCESS ROAD
PLAN, PROFILE AND SECTION
STA. 36+10 TO STA. 46+00

SUBMITTED BY:
DATE APPROVED:
SPEC. NO. DACW09- ____ -B- ____
DISTRICT FILE NO.
SHEET 4 OF 5

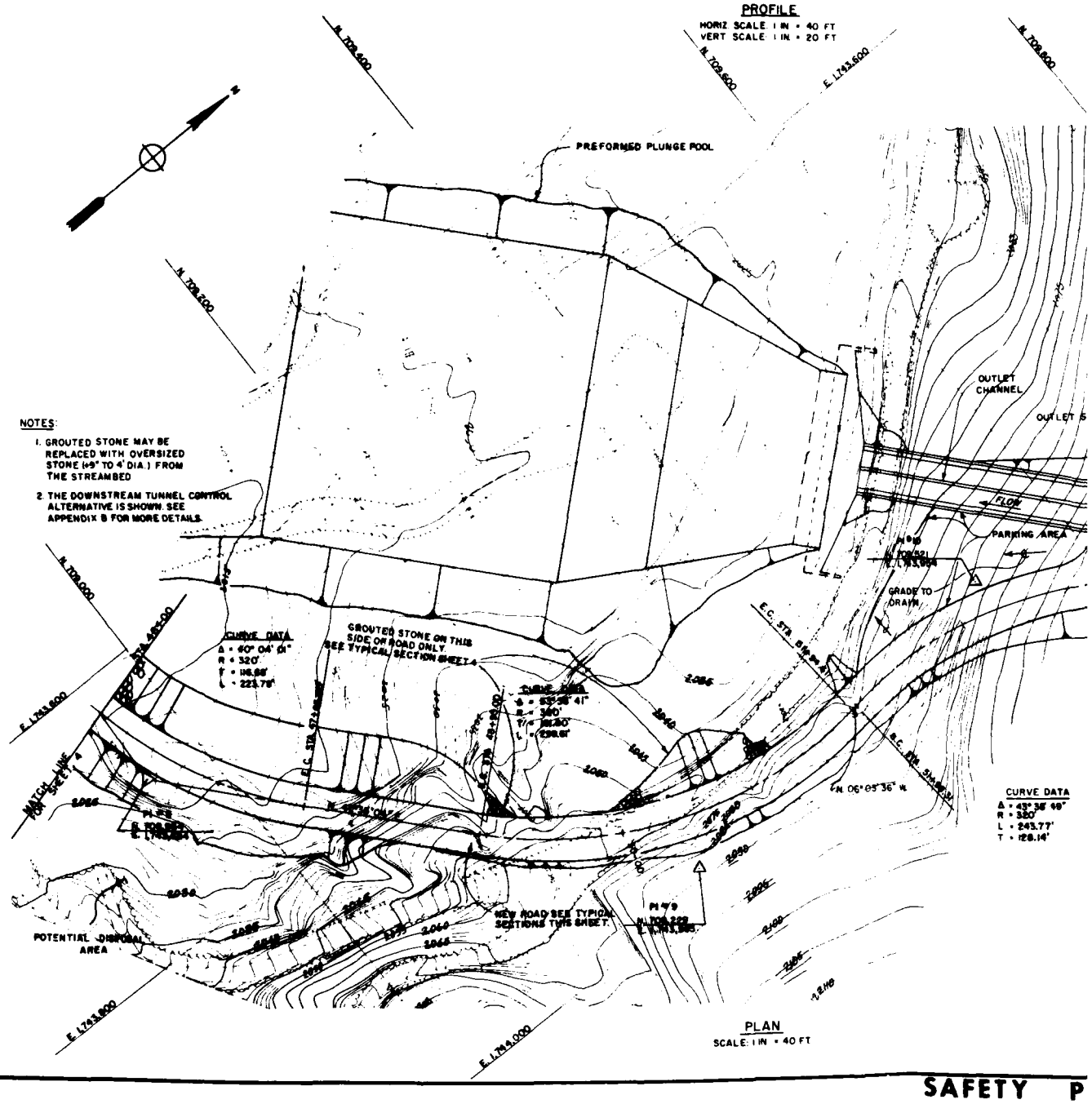
PLATE 14

SAFETY PAYS



PROFILE
HORIZ SCALE: 1 IN = 40 FT
VERT SCALE: 1 IN = 20 FT

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

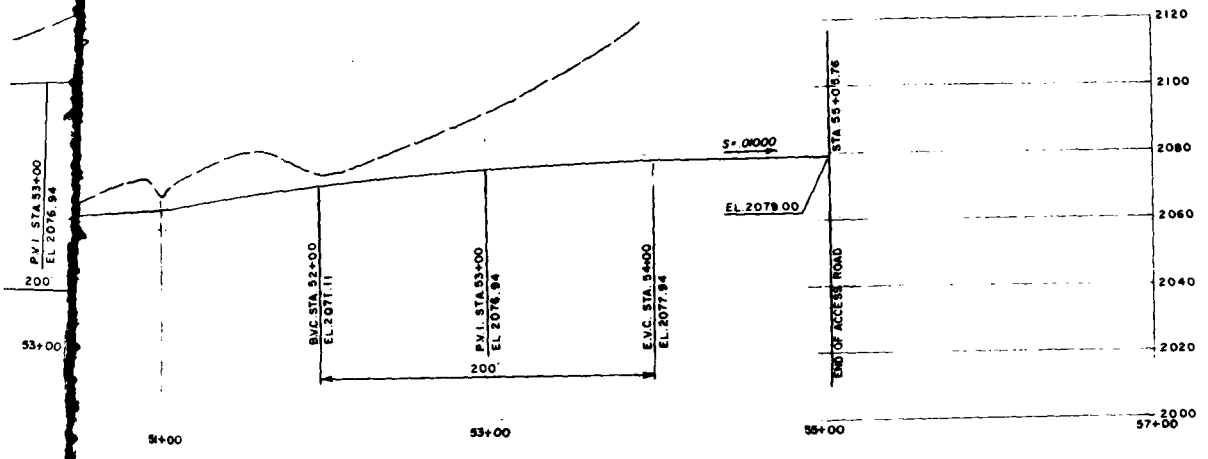


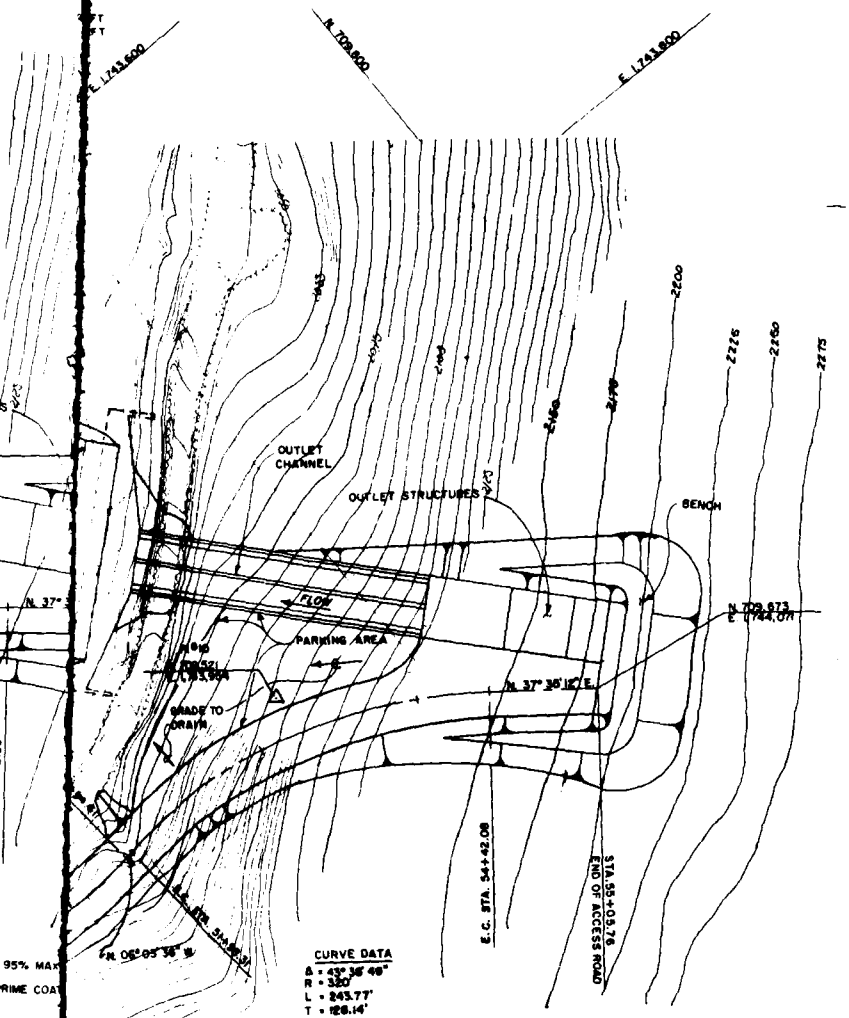
NOTES:

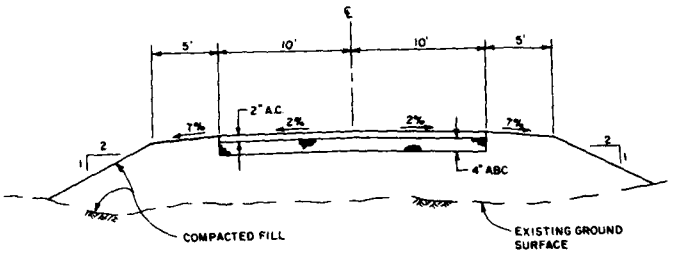
1. GROUTED STONE MAY BE REPLACED WITH OVERSIZED STONE (+9" TO 4' DIA.) FROM THE STREAMBED
2. THE DOWNSTREAM TUNNEL CONTROL ALTERNATIVE IS SHOWN. SEE APPENDIX B FOR MORE DETAILS.

PLAN
SCALE: 1 IN = 40 FT

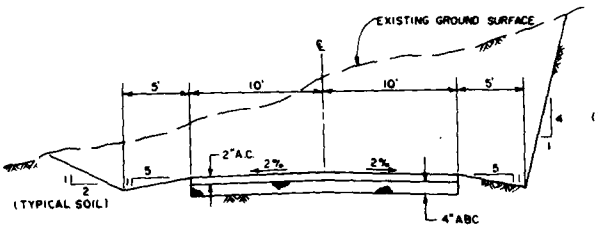
VALUE ENGINEERING PAYS



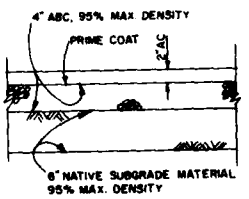




TYPICAL FILL SECTION
NOT TO SCALE



TYPICAL CUT SECTION
NOT TO SCALE



TYPICAL PAVEMENT SECTION
NOT TO SCALE

SCALE: 1 IN. = 40 FT
50 0 50 100
SCALE: 1 IN. = 20 FT.
10 0 10 20 30 40 50

DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

SYMBOL	DESCRIPTION	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY: JAA P.PENNEKER
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

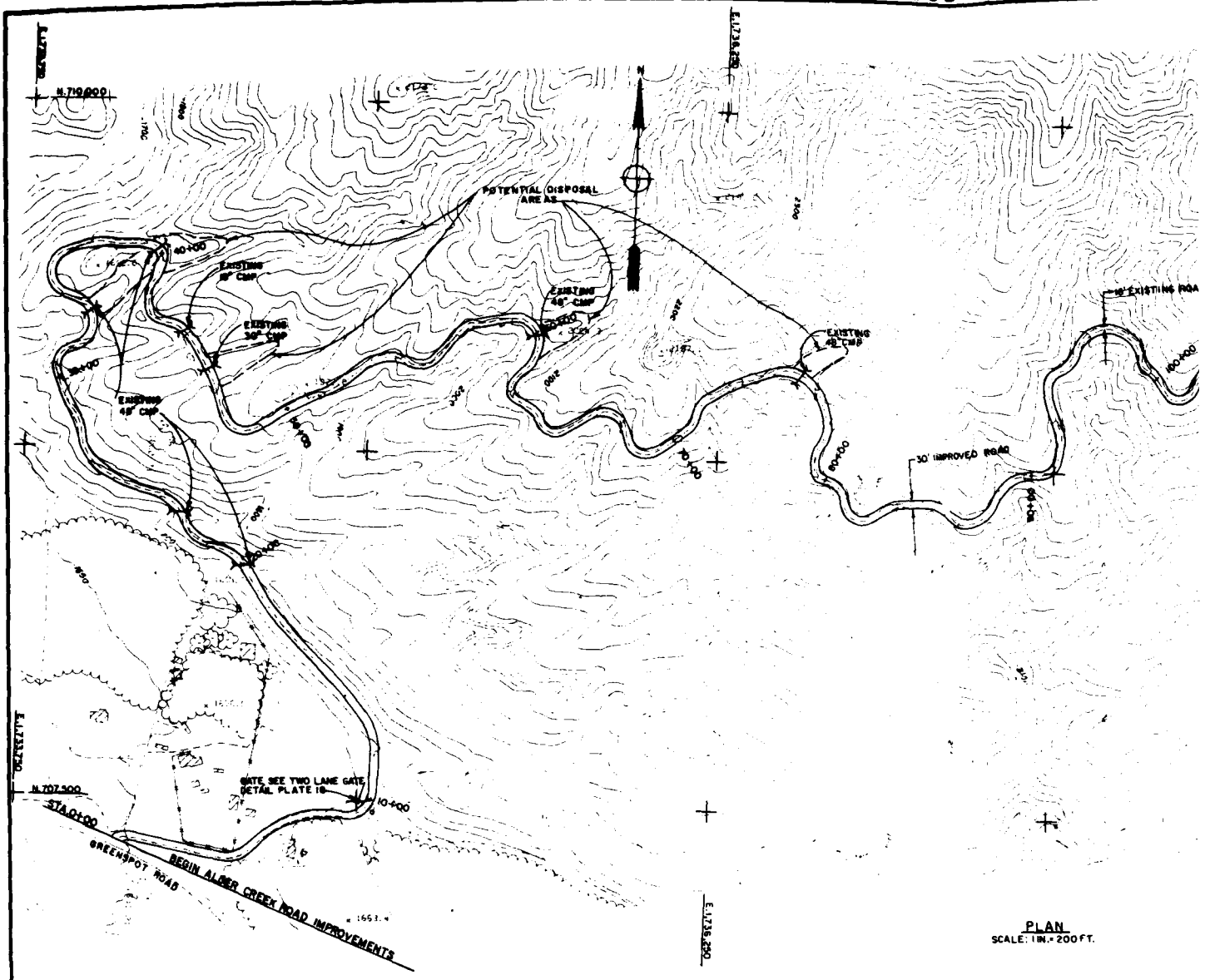
SEVEN OAKS DAM

OUTLET WORKS ACCESS ROAD
PLAN, PROFILE AND SECTIONS
STA 46+00 TO STA. 54+06.87

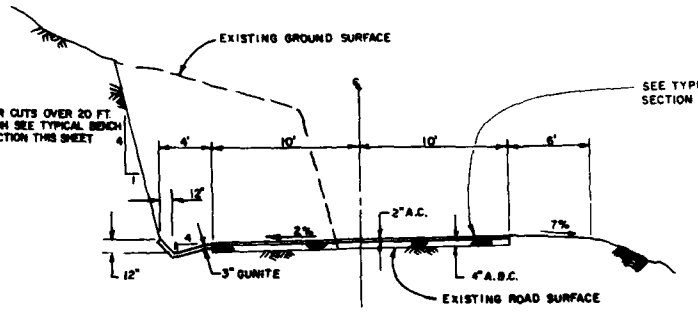
SUBMITTED BY:
DATE APPROVED:
SPEC. NO. DACW09- ____ B- ____
DISTRICT FILE NO.
SHEET 5 OF 5

PLATE I

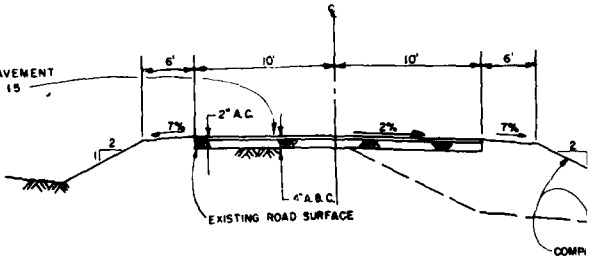
SAFETY PAYS



PLAN
SCALE: 1 IN. = 200 FT.

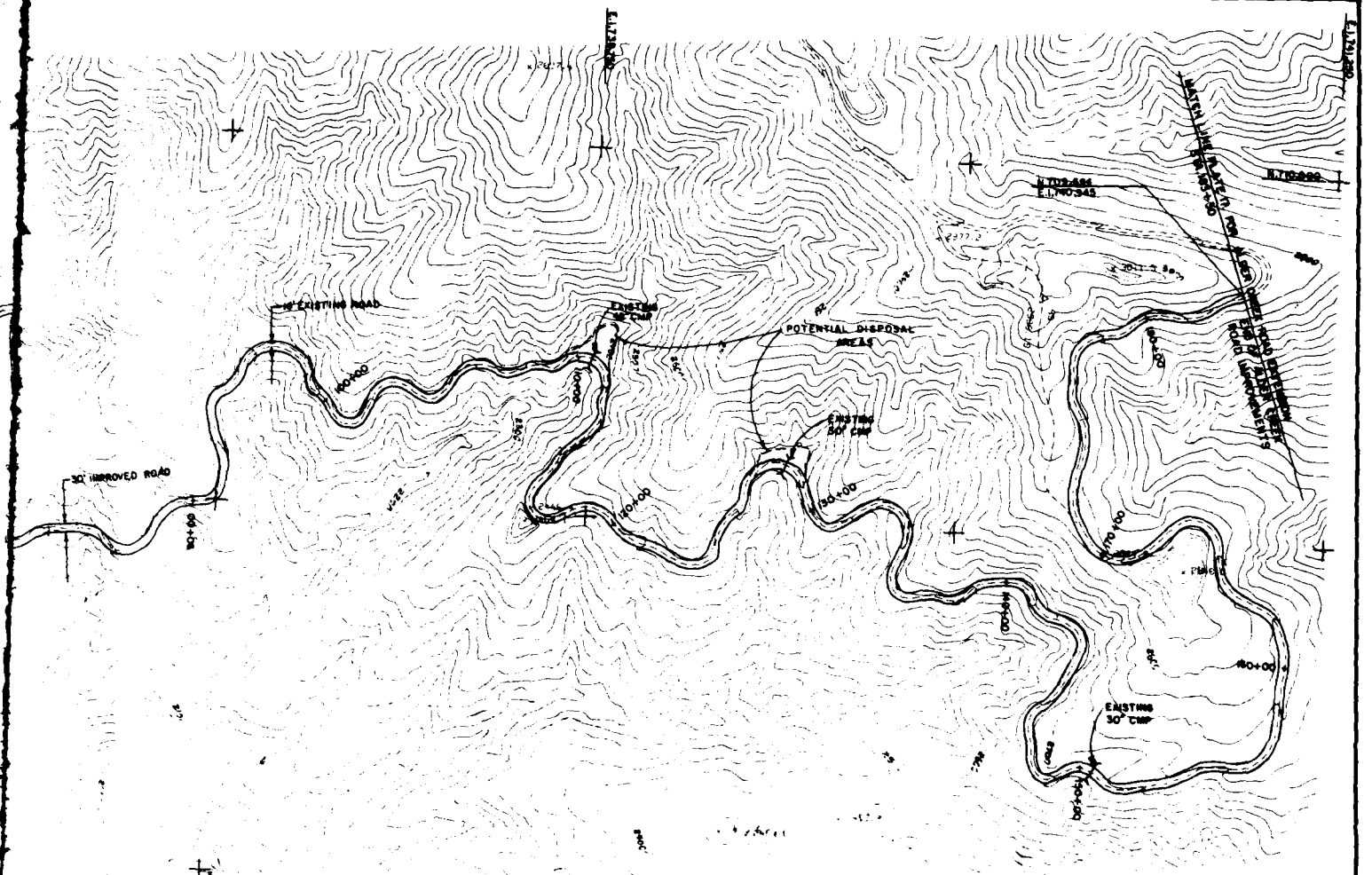


TYPICAL CUT SECTION
NOT TO SCALE

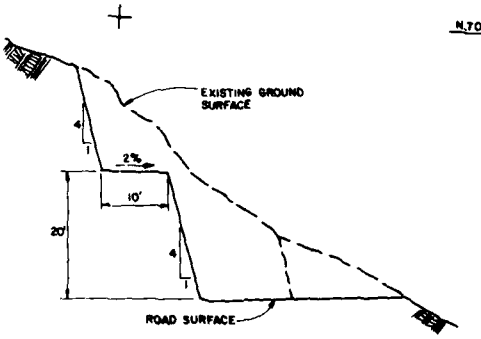


TYPICAL FILL SECTION
NOT TO SCALE

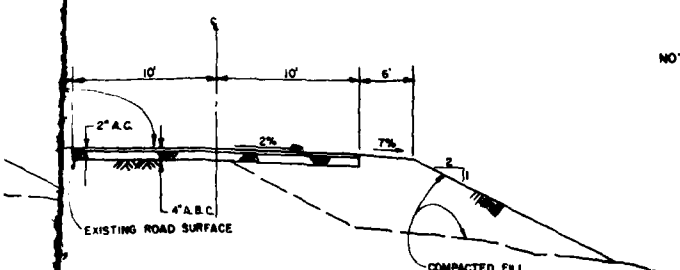
ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING



PLAN
SCALE: 1 IN. = 200 FT.



TYPICAL BENCH SECTION
NOT TO SCALE



TYPICAL FILL SECTION
NOT TO SCALE

NOTES:
1. EXISTING CORRUGATED METAL PIPE SHALL BE LENGTHENED TO ACCOMMODATE WIDENED ROAD.
2. GUARDRAILING SHALL BE PLACED ON ALL HORIZONTAL CURVES WHERE THE FRONT SLOPES DESCEND MORE THAN 8 FEET. SEE DETAILS PLATE 16. GUIDEPOSTS SHALL BE PLACED ON ALL STRAIGHTAWAYS WHERE THE FRONT SLOPE DESCENDS MORE THAN 10 FEET.
3. TURNOUTS 10 FEET WIDE AND NOT LONGER THAN 50 FEET SHALL BE SPACED IN INTERVALS OF 500 TO 1000 FEET.

SCALE: 1 IN. = 200 FT.
100 0 100 200 300 400 500

DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
REVISIONS			

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

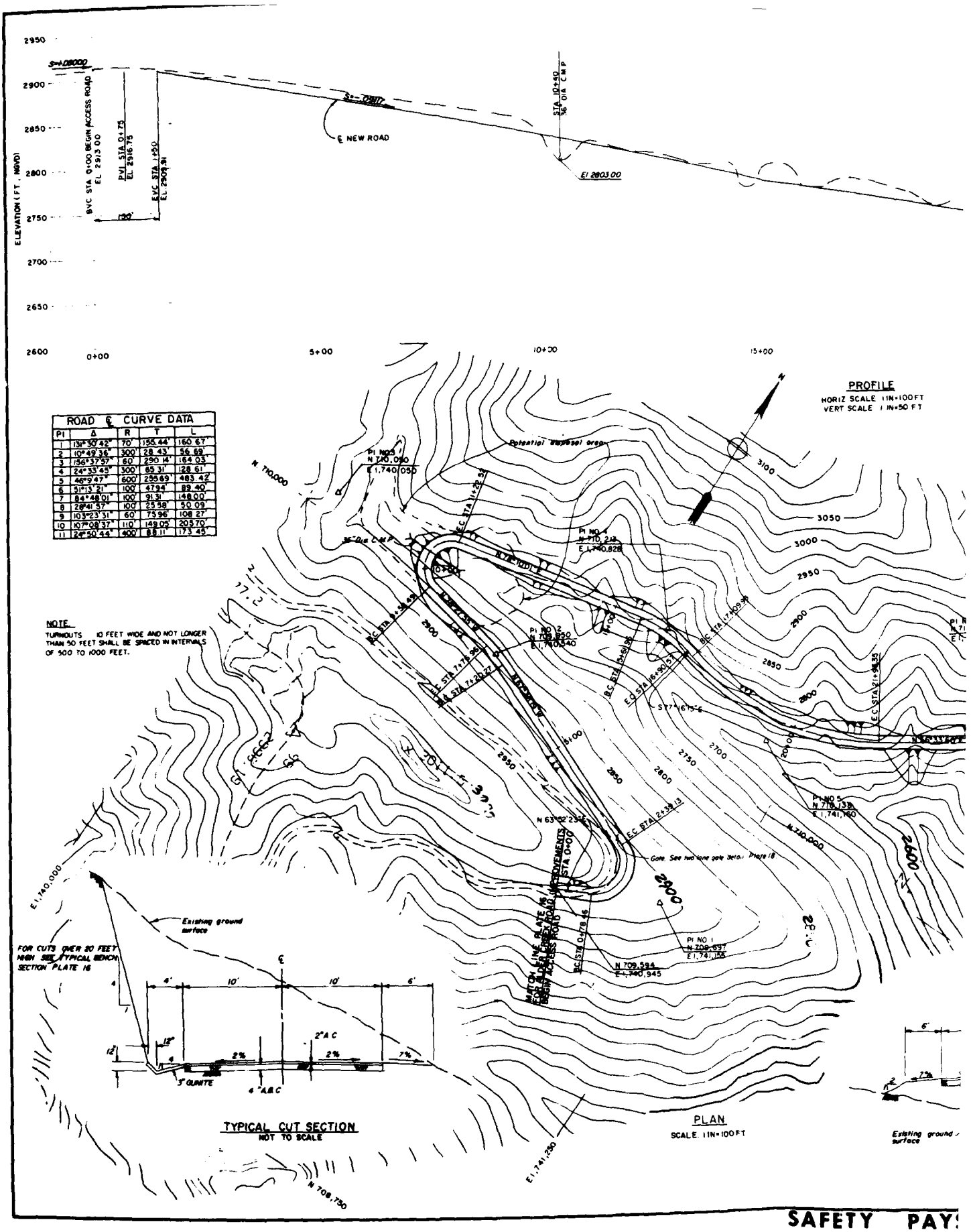
DESIGNED BY:
DRAWN BY: D. VILPPU
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

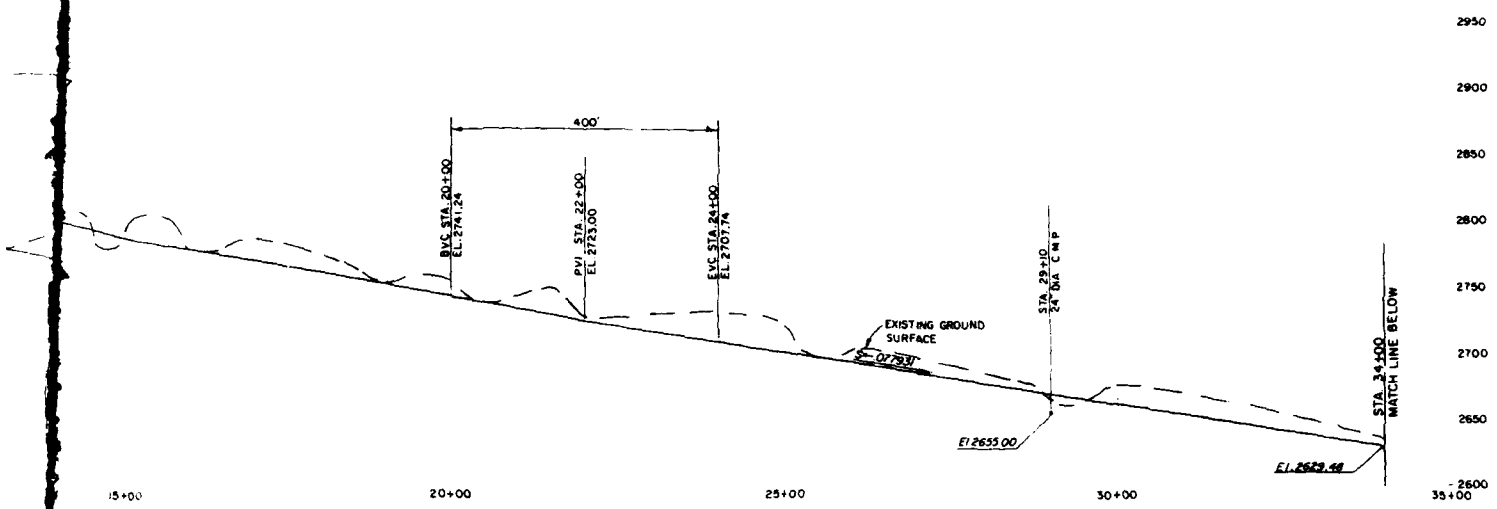
SEVEN OAKS DAM
ALDER CREEK ROAD IMPROVEMENT
PLAN AND SECTIONS

SUBMITTED BY: DATE APPROVED:
CHIEF, BRANCH
SPEC. NO. SACW09-____-B-____
DISTRICT FILE NO.
SHEET 1 OF 1

PLATE 16


PROFILE
HORIZ SCALE 1 IN=100 FT
VERT SCALE 1 IN=50 FT
ELEVATION (FT., NGVD)
2950
2900
2850
2800
2750
2700
2650
2600
0+00
5+00
10+00
15+00
BVC STA 0+00 BEGIN ACCESS ROAD EL 2913.00
PVI STA 0+75 EL 2916.75
EVC STA 1+50 EL 2909.91
S=+08000
S=-0317
℄ NEW ROAD
STA 10+40 36" DIA CMP
El 2803.00
ROAD ℄ CURVE DATA
PI | Δ | R | T | L
1 | 131°30'42" | 70' | 155.44' | 160.67'
2 | 10°49'36" | 300' | 28.43' | 56.69'
3 | 156°37'57" | 60' | 290.14' | 164.03'
4 | 24°33'48" | 300' | 65.31' | 128.61'
5 | 46°9'47" | 600' | 256.69' | 483.42'
6 | 51°13'21" | 100' | 47.94' | 89.40'
7 | 84°48'01" | 100' | 91.31' | 148.00'
8 | 28°41'57" | 100' | 25.58' | 50.09'
9 | 103°23'31" | 60' | 75.96' | 108.27'
10 | 107°08'37" | 110' | 149.05' | 205.70'
11 | 24°50'44" | 400' | 88.11' | 173.45'
NOTE
TURNOUTS 10 FEET WIDE AND NOT LONGER THAN 50 FEET SHALL BE SPACED IN INTERVALS OF 500 TO 1000 FEET.
ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING
N 710,000
E 1,740,000
E 1,741,250
N 708,750
Potential disposal area
PI NO 3
N 710,090
E 1,740,050
PI NO 4
N 710,249
E 1,740,828
PI NO 2
N 709,850
E 1,740,540
PI NO 5
N 710,138
E 1,741,160
PI NO 1
N 709,697
E 1,741,145
N 709,594
E 1,740,945
BC STA 0+78.46
EC STA 1+29.51
BC STA 1+49.39
EC STA 2+38.13
BC STA 3+61.95
EC STA 4+09.35
BC STA 5+34.98
EC STA 6+50.51
BC STA 7+78.38
EC STA 7+90.27
EC STA 21+94.35
N 63°52'23"E
S 17°16'15"E
N 20°11'32"E
N 38°22'
Gate. See two lane gate detail Plate 18
MATCH LINE PLATE 16
IMPROVEMENTS BEGIN ACCESS ROAD STA 0+00
FOR CUTS OVER 20 FEET HIGH SEE TYPICAL BENCH SECTION PLATE 16
Existing ground surface
℄
4'
10'
10'
6'
4
1
12"
12"
2%
2%
7%
2" A C
3" GUNITE
4 " A.B.C
TYPICAL CUT SECTION
NOT TO SCALE
PLAN
SCALE 1 IN=100 FT
Existing ground surface

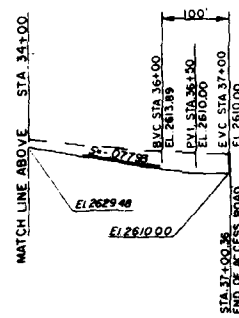
SAFETY PAY



PROFILE
HORIZ SCALE 1 IN=100 FT
VERT SCALE 1 IN=50 FT

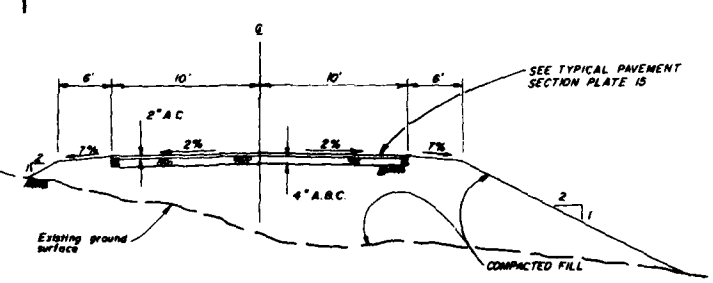
GUARDRAIL STATIONING	
LEFT SIDE OF ROAD	RIGHT SIDE OF ROAD
9+58 TO 11+23	0+78 TO 2+39
28+63 TO 29+71	7+20 TO 7+77
	9+58 TO 11+23
	15+62 TO 21+94
	24+47 TO 29+71
	32+70 TO 36+50

GUIDEPOST STATIONING	
LEFT SIDE OF ROAD	RIGHT SIDE OF ROAD
	0+00 TO 0+78
	2+39 TO 7+20
	7+77 TO 9+58
	11+23 TO 15+62
	21+94 TO 24+47
	29+71 TO 32+70



SCALE 1 IN = 100 FT
100 0 100 200 300
SCALE 1 IN. = 50 FT
50 0 50 100 150

DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929



TYPICAL FILL SECTION
NOT TO SCALE

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY: D. VILPPU
PLANS/SPECS BY:
CHECKED BY:

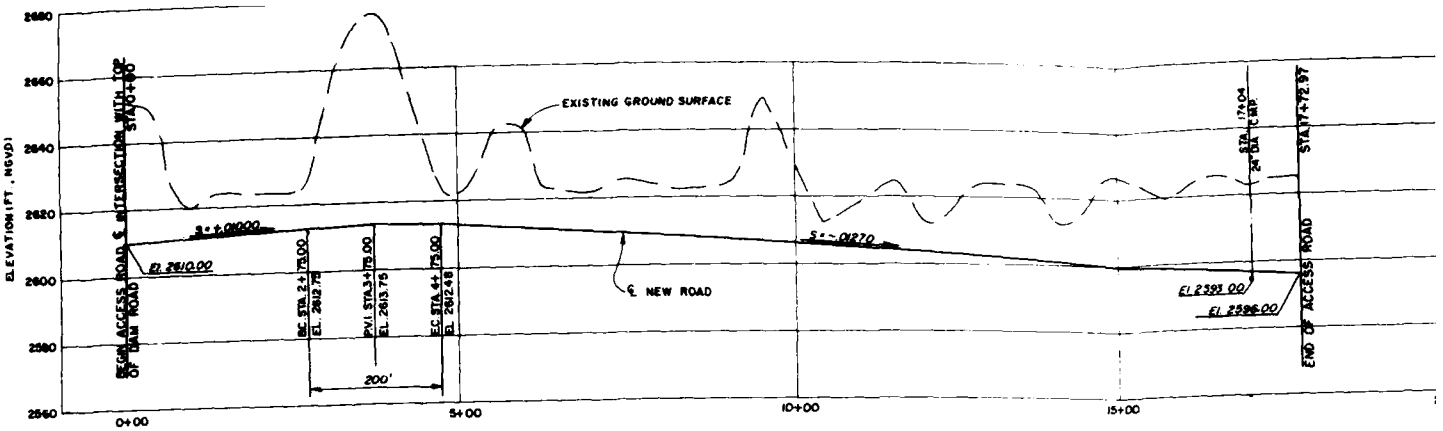
SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

ALDER CREEK ROAD EXTENSION
PLAN, PROFILE AND SECTIONS

SUBMITTED BY:
DATE APPROVED:
SPEC. NO. DACW09- - -
DISTRICT FILE NO.
SHEET 1 OF 1

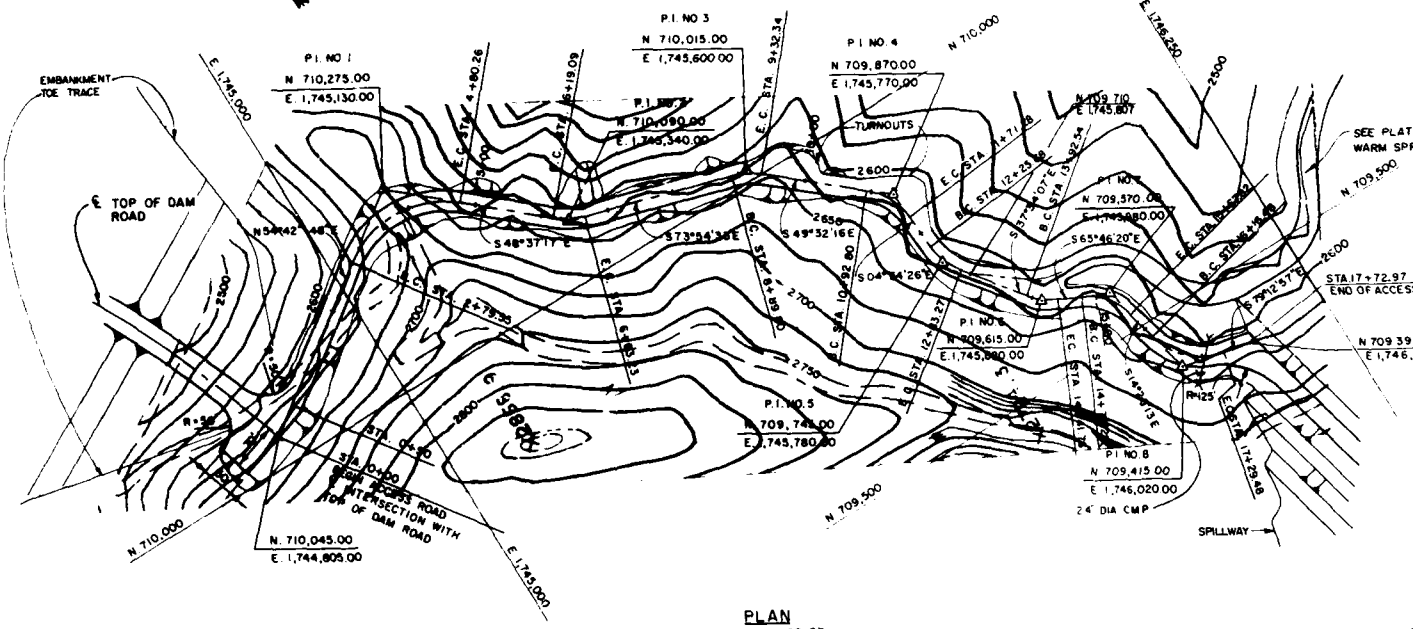
PLATE 17



PROFILE

VERT. SCALE: 1 IN. = 20 FT.
HORIZ. SCALE: 1 IN. = 100 FT.

ROAD ℄ CURVE DATA				
PI	△	R	T	L
1	76°39'55"	150	118.60'	200.71
2	25°17'16"	100'	22.43'	44.14
3	24°22'17"	100'	21.59'	42.54'
4	44°57'50"	100	41.38'	78.46'
5	52°59'41"	100'	29.62'	57.59'
6	28°12'13"	100'	25.12'	49.22'
7	51°18'07"	100'	48.02'	89.54'
8	64°44'43"	100'	63.40'	113.00'



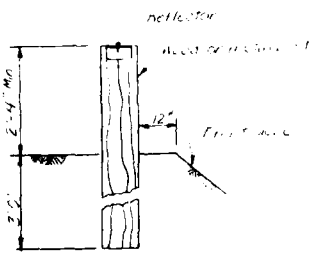
PLAN

SCALE: 1 IN. = 100 FT.

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

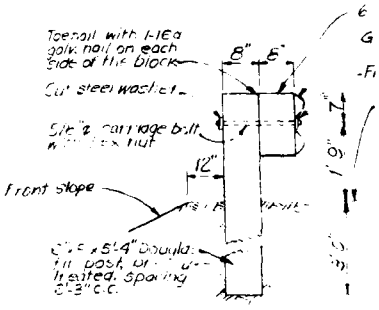
NOTES:

1. SEE SHEET 17 FOR TYPICAL CROSS SECTIONS.
2. GUARDRAILING SHALL BE PLACED ON THE LEFT SIDE OF THE ROAD ONLY FROM STA. 8+90 TO STA. 17+73. GUIDEPOSTS SHALL BE PLACED ON THE LEFT OF THE ROAD ONLY FROM STA. 0+80 TO STA. 2+80 AND FROM STA. 4+40 TO STA. 8+90. SEE DETAILS THIS SHEET.



GUIDEPOST DETAIL

NOT TO SCALE

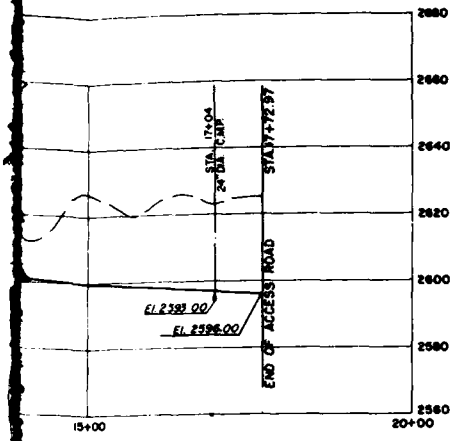


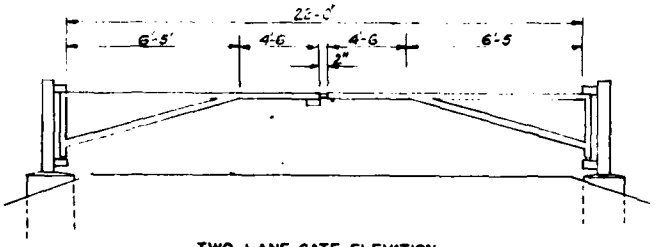
GUARDRAIL DETAIL

NOT TO SCALE

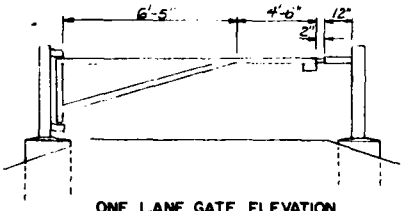
SAFETY PAYS

VALUE ENGINEERING PAYS



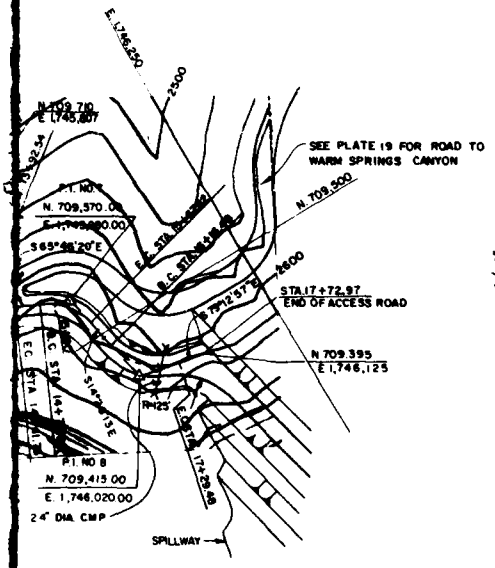


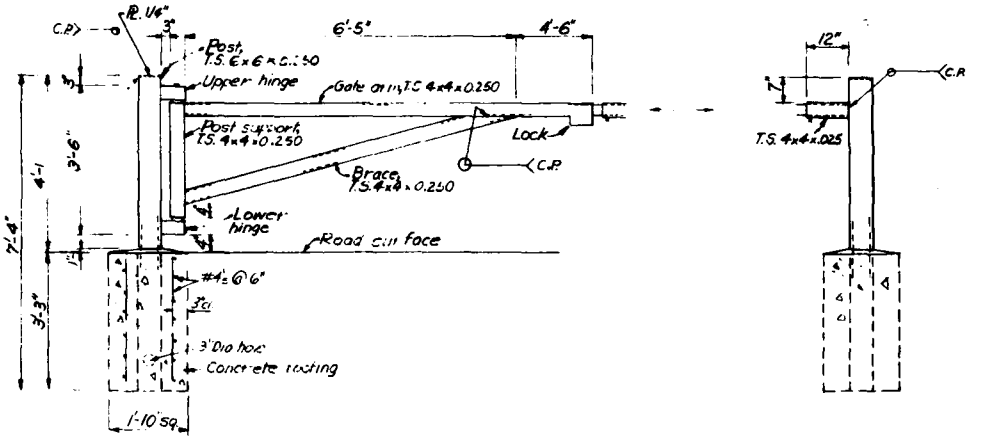
TWO LANE GATE ELEVATION
NOT TO SCALE



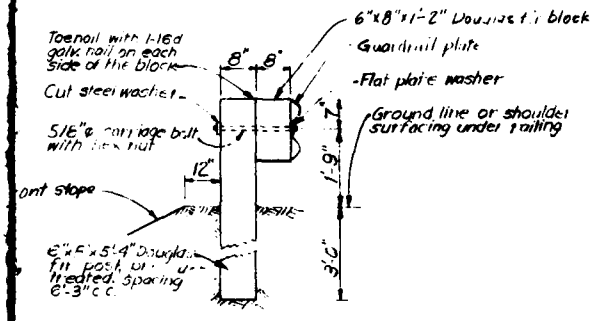
ONE LANE GATE ELEVATION
NOT TO SCALE

ROAD ℄ CURVE DATA				
PI	△	R	T	L
1	76°39'55"	150	118 60'	200 71
2	25°17'16"	100'	22 43'	44 14
3	24°22'17"	100'	21 59'	42 54'
4	44°5'50"	100	41 38'	76 48'
5	32°59'41"	100'	29 62'	57 59'
6	28°12'13"	100'	25.12'	49.22'
7	51°18'07"	100'	48.02'	89.54'
8	64°44'43"	100'	63.49'	113.00'





GATE DETAILS
NOT TO SCALE



GUARDRAIL DETAIL
NOT TO SCALE

SCALE: 1 IN. = 20 FT.
20 0 20 30 60

SCALE: 1 IN. = 100 FT.
100 0 100 200 300

DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

SYMBOL	DESIGNATION	DATE	APPROVAL
REVISIONS			

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY: GBA
CHECKED BY:

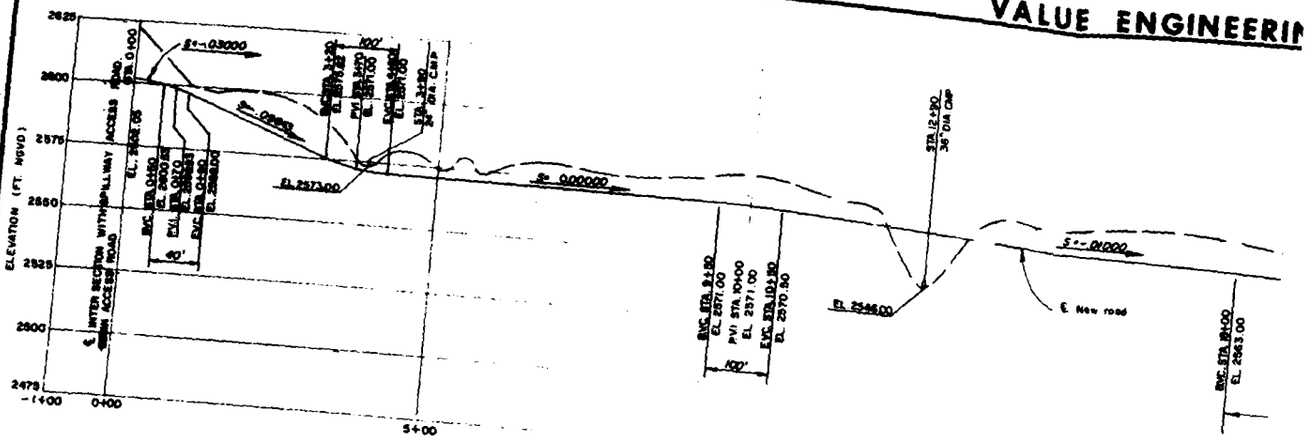
SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM
TOP OF DAM TO SPILLWAY ACCESS ROAD
PLAN, PROFILE AND DETAILS.

SUBMITTED BY: DATE APPROVED:
SPEC. NO. DACW09-____-B-____
DISTRICT FILE NO.
SHEET 1 OF 1

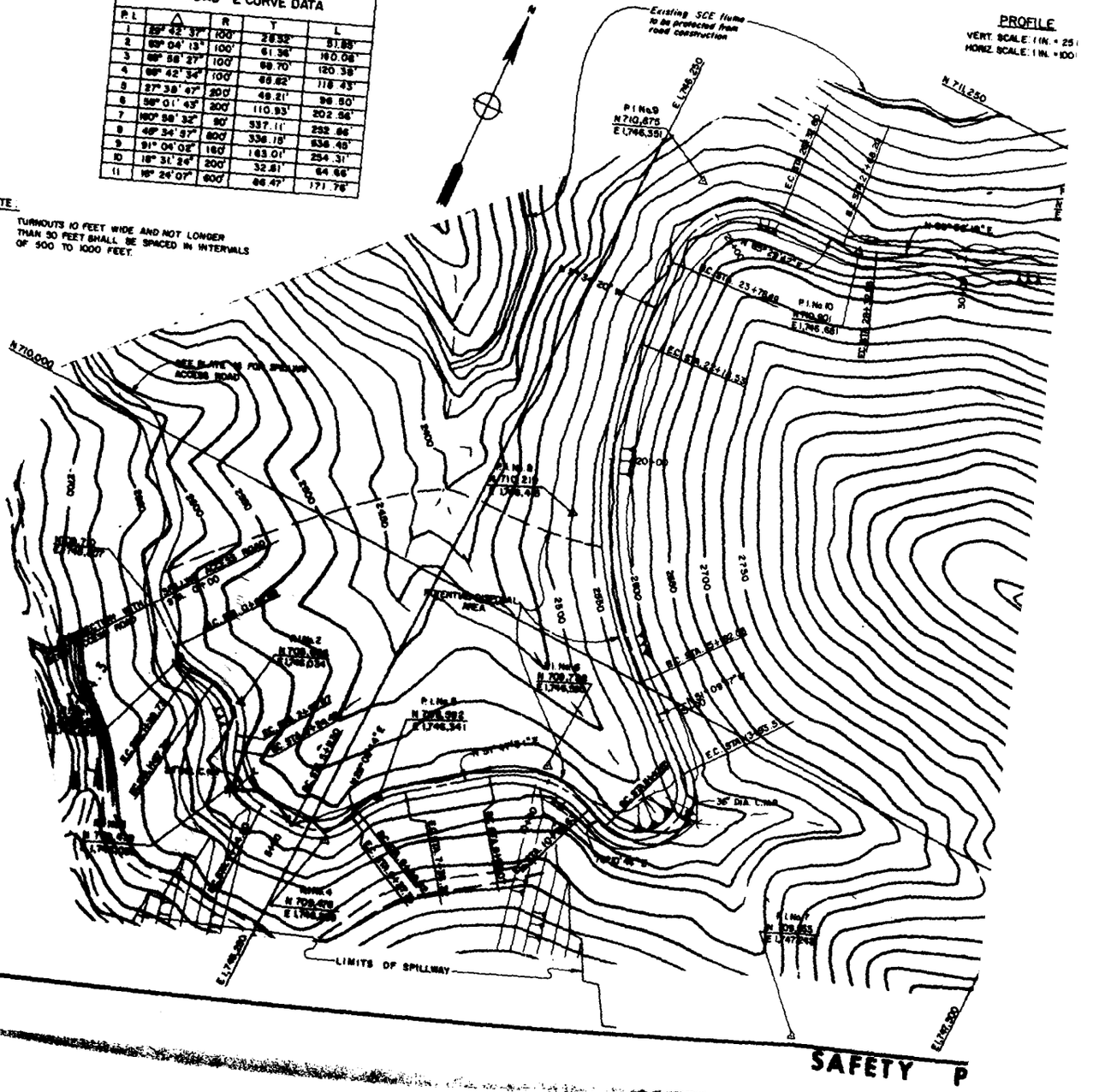
PLATE 18

SAFETY PAYS



ROAD ℄ CURVE DATA				
P.I.	Δ	R	T	L
1	35° 32' 37"	100'	28.32'	61.88'
2	69° 04' 13"	100'	61.36'	120.56'
3	69° 56' 27"	100'	69.70'	120.38'
4	68° 42' 34"	100'	68.62'	118.43'
5	27° 39' 47"	200'	49.21'	96.50'
6	58° 01' 43"	200'	110.93'	202.56'
7	180° 58' 32"	90'	337.11'	283.66'
8	48° 34' 57"	800'	336.18'	636.48'
9	91° 04' 02"	180'	183.01'	284.31'
10	18° 31' 24"	200'	32.81'	64.66'
11	16° 24' 07"	600'	86.47'	171.76'

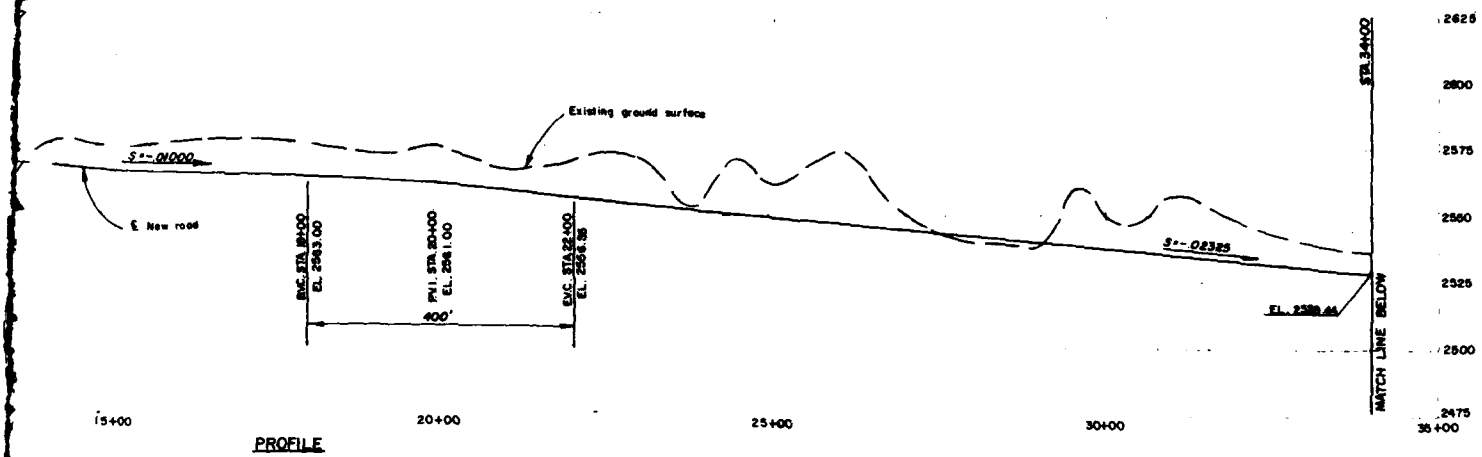
NOTE :
TURNOUTS 10 FEET WIDE AND NOT LONGER THAN 30 FEET SHALL BE SPACED IN INTERVALS OF 500 TO 1000 FEET.



ENVIRONMENTAL
ENHANCEMENT
THRU ENGINEERING

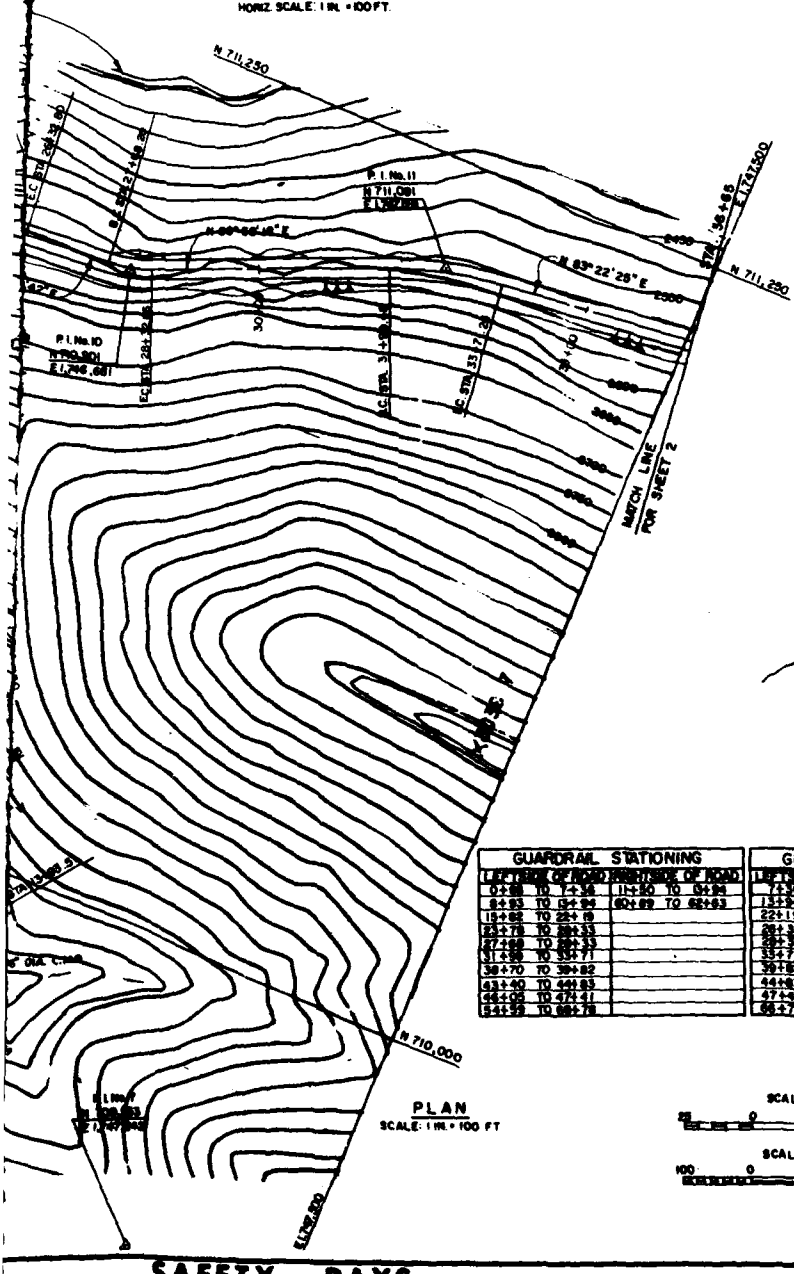
SAFETY P

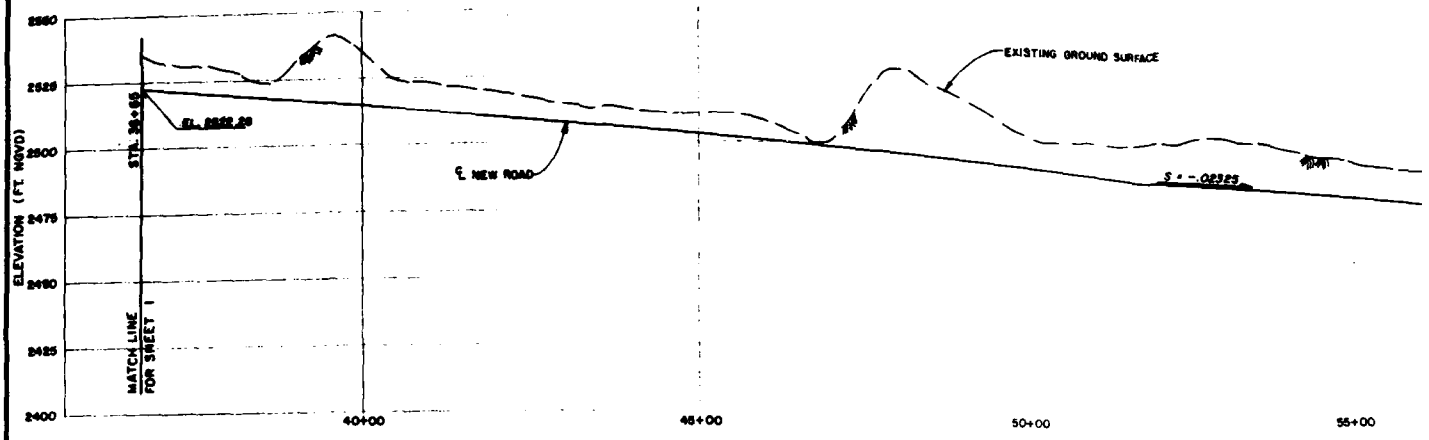
VALUE ENGINEERING PAYS



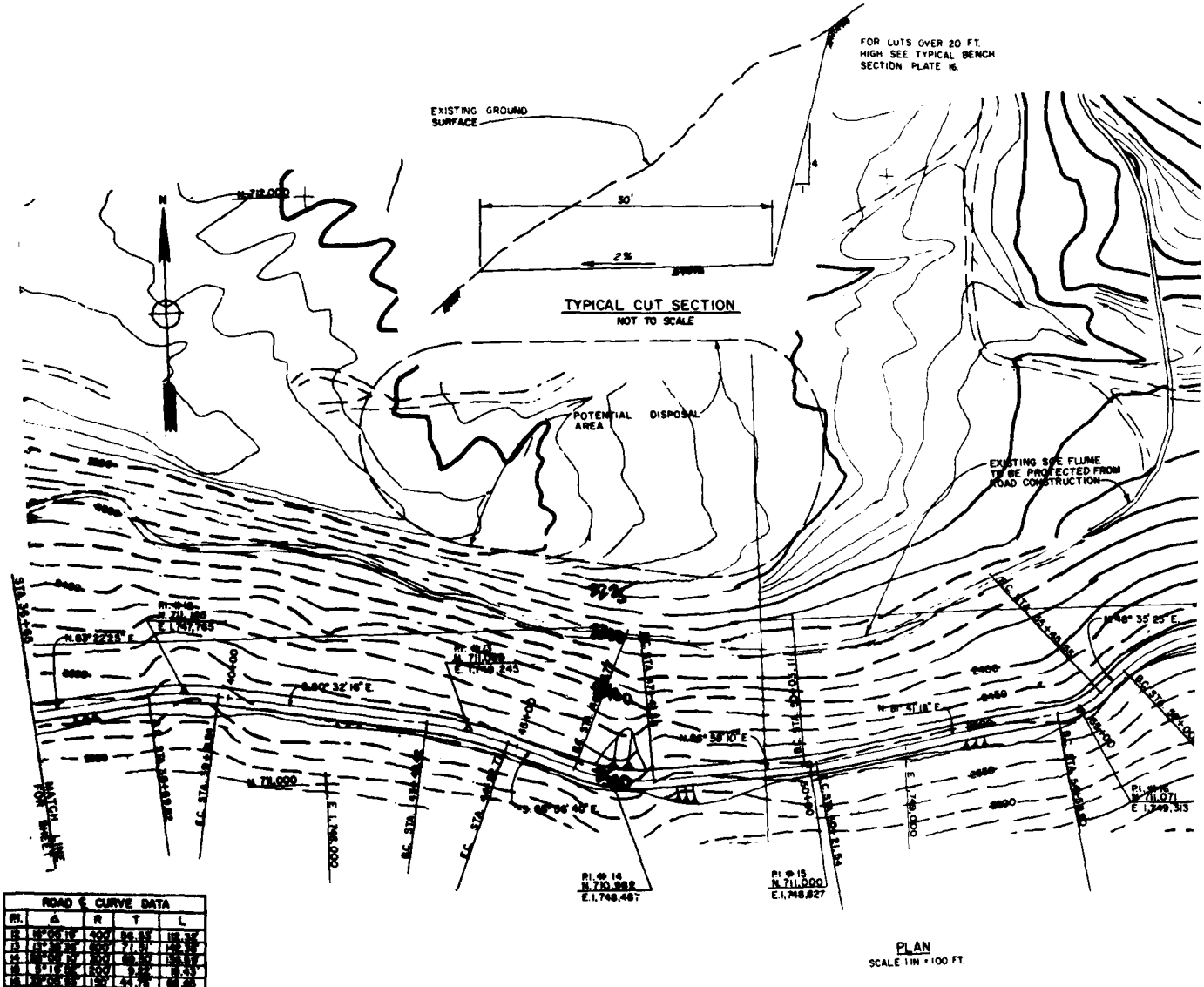
PROFILE

VERT. SCALE: 1 IN. = 25 FT.
HORIZ. SCALE: 1 IN. = 100 FT.


N 711,250
P.I. No. 11
N 711,091
E 1,747,500
N 65°22'28" E
N 711,250
P.I. No. 10
N 710,801
E 1,746,681
N 66°46'18" E
MATCH LINE FOR SHEET 2
N 710,000
℄ New road
EL. 2560.00
EL. 2556.18
EL. 2553.00
EL. 2551.00
EL. 2548.81
EL. 2546.68
EL. 2543.50
EL. 2541.50
EL. 2538.31
EL. 2536.31
EL. 2533.12
EL. 2531.12
EL. 2527.93
EL. 2525.93
EL. 2522.74
EL. 2520.74
EL. 2517.55
EL. 2515.55
EL. 2512.36
EL. 2510.36
EL. 2507.17
EL. 2505.17
EL. 2502.00
EL. 2500.00
EL. 2496.80
EL. 2494.80
EL. 2491.61
EL. 2489.61
EL. 2486.42
EL. 2484.42
EL. 2481.23
EL. 2479.23
EL. 2476.04
EL. 2474.04
EL. 2470.85
EL. 2468.85
EL. 2465.66
EL. 2463.66
EL. 2460.47
EL. 2458.47
EL. 2455.28
EL. 2453.28
EL. 2450.09
EL. 2448.09
EL. 2444.90
EL. 2442.90
EL. 2439.71
EL. 2437.71
EL. 2434.52
EL. 2432.52
EL. 2429.33
EL. 2427.33
EL. 2424.14
EL. 2422.14
EL. 2418.95
EL. 2416.95
EL. 2413.76
EL. 2411.76
EL. 2408.57
EL. 2406.57
EL. 2403.38
EL. 2401.38
EL. 2398.19
EL. 2396.19
EL. 2393.00
EL. 2391.00
EL. 2387.81
EL. 2385.81
EL. 2382.62
EL. 2380.62
EL. 2377.43
EL. 2375.43
EL. 2372.24
EL. 2370.24
EL. 2367.05
EL. 2365.05
EL. 2361.86
EL. 2359.86
EL. 2356.67
EL. 2354.67
EL. 2351.48
EL. 2349.48
EL. 2346.29
EL. 2344.29
EL. 2341.10
EL. 2339.10
EL. 2335.91
EL. 2333.91
EL. 2330.72
EL. 2328.72
EL. 2325.53
EL. 2323.53
EL. 2320.34
EL. 2318.34
EL. 2315.15
EL. 2313.15
EL. 2309.96
EL. 2307.96
EL. 2304.77
EL. 2302.77
EL. 2299.58
EL. 2297.58
EL. 2294.39
EL. 2292.39
EL. 2289.20
EL. 2287.20
EL. 2284.01
EL. 2282.01
EL. 2278.82
EL. 2276.82
EL. 2273.63
EL. 2271.63
EL. 2268.44
EL. 2266.44
EL. 2263.25
EL. 2261.25
EL. 2258.06
EL. 2256.06
EL. 2252.87
EL. 2250.87
EL. 2247.68
EL. 2245.68
EL. 2242.49
EL. 2240.49
EL. 2237.30
EL. 2235.30
EL. 2232.11
EL. 2230.11
EL. 2226.92
EL. 2224.92
EL. 2221.73
EL. 2219.73
EL. 2216.54
EL. 2214.54
EL. 2211.35
EL. 2209.35
EL. 2206.16
EL. 2204.16
EL. 2200.97
EL. 2198.97
EL. 2195.78
EL. 2193.78
EL. 2190.59
EL. 2188.59
EL. 2185.40
EL. 2183.40
EL. 2180.21
EL. 2178.21
EL. 2175.02
EL. 2173.02
EL. 2169.83
EL. 2167.83
EL. 2164.64
EL. 2162.64
EL. 2159.45
EL. 2157.45
EL. 2154.26
EL. 2152.26
EL. 2149.07
EL. 2147.07
EL. 2143.88
EL. 2141.88
EL. 2138.69
EL. 2136.69
EL. 2133.50
EL. 2131.50
EL. 2128.31
EL. 2126.31
EL. 2123.12
EL. 2121.12
EL. 2117.93
EL. 2115.93
EL. 2112.74
EL. 2110.74
EL. 2107.55
EL. 2105.55
EL. 2102.36
EL. 2100.36
EL. 2097.17
EL. 2095.17
EL. 2091.98
EL. 2089.98
EL. 2086.79
EL. 2084.79
EL. 2081.60
EL. 2079.60
EL. 2076.41
EL. 2074.41
EL. 2071.22
EL. 2069.22
EL. 2066.03
EL. 2064.03
EL. 2060.84
EL. 2058.84
EL. 2055.65
EL. 2053.65
EL. 2050.46
EL. 2048.46
EL. 2045.27
EL. 2043.27
EL. 2040.08
EL. 2038.08
EL. 2034.89
EL. 2032.89
EL. 2029.70
EL. 2027.70
EL. 2024.51
EL. 2022.51
EL. 2019.32
EL. 2017.32
EL. 2014.13
EL. 2012.13
EL. 2008.94
EL. 2006.94
EL. 2003.75
EL. 2001.75
EL. 1998.56
EL. 1996.56
EL. 1993.37
EL. 1991.37
EL. 1988.18
EL. 1986.18
EL. 1982.99
EL. 1980.99
EL. 1977.80
EL. 1975.80
EL. 1972.61
EL. 1970.61
EL. 1967.42
EL. 1965.42
EL. 1962.23
EL. 1960.23
EL. 1957.04
EL. 1955.04
EL. 1951.85
EL. 1949.85
EL. 1946.66
EL. 1944.66
EL. 1941.47
EL. 1939.47
EL. 1936.28
EL. 1934.28
EL. 1931.09
EL. 1929.09
EL. 1925.90
EL. 1923.90
EL. 1920.71
EL. 1918.71
EL. 1915.52
EL. 1913.52
EL. 1910.33
EL. 1908.33
EL. 1905.14
EL. 1903.14
EL. 1899.95
EL. 1897.95
EL. 1894.76
EL. 1892.76
EL. 1889.57
EL. 1887.57
EL. 1884.38
EL. 1882.38
EL. 1879.19
EL. 1877.19
EL. 1874.00
EL. 1872.00
EL. 1868.81
EL. 1866.81
EL. 1863.62
EL. 1861.62
EL. 1858.43
EL. 1856.43
EL. 1853.24
EL. 1851.24
EL. 1848.05
EL. 1846.05
EL. 1842.86
EL. 1840.86
EL. 1837.67
EL. 1835.67
EL. 1832.48
EL. 1830.48
EL. 1827.29
EL. 1825.29
EL. 1822.10
EL. 1820.10
EL. 1816.91
EL. 1814.91
EL. 1811.72
EL. 1809.72
EL. 1806.53
EL. 1804.53
EL. 1801.34
EL. 1799.34
EL. 1796.15
EL. 1794.15
EL. 1790.96
EL. 1788.96
EL. 1785.77
EL. 1783.77
EL. 1780.58
EL. 1778.58
EL. 1775.39
EL. 1773.39
EL. 1770.20
EL. 1768.20
EL. 1765.01
EL. 1763.01
EL. 1759.82
EL. 1757.82
EL. 1754.63
EL. 1752.63
EL. 1749.44
EL. 1747.44
EL. 1744.25
EL. 1742.25
EL. 1739.06
EL. 1737.06
EL. 1733.87
EL. 1731.87
EL. 1728.68
EL. 1726.68
EL. 1723.49
EL. 1721.49
EL. 1718.30
EL. 1716.30
EL. 1713.11
EL. 1711.11
EL. 1707.92
EL. 1705.92
EL. 1702.73
EL. 1700.73
EL. 1697.54
EL. 1695.54
EL. 1692.35
EL. 1690.35
EL. 1687.16
EL. 1685.16
EL. 1681.97
EL. 1679.97
EL. 1676.78
EL. 1674.78
EL. 1671.59
EL. 1669.59
EL. 1666.40
EL. 1664.40
EL. 1661.21
EL. 1659.21
EL. 1656.02
EL. 1654.02
EL. 1650.83
EL. 1648.83
EL. 1645.64
EL. 1643.64
EL. 1640.45
EL. 1638.45
EL. 1635.26
EL. 1633.26
EL. 1630.07
EL. 1628.07
EL. 1624.88
EL. 1622.88
EL. 1619.69
EL. 1617.69
EL. 1614.50
EL. 1612.50
EL. 1609.31
EL. 1607.31
EL. 1604.12
EL. 1602.12
EL. 1598.93
EL. 1596.93
EL. 1593.74
EL. 1591.74
EL. 1588.55
EL. 1586.55
EL. 1583.36
EL. 1581.36
EL. 1578.17
EL. 1576.17
EL. 1572.98
EL. 1570.98
EL. 1567.79
EL. 1565.79
EL. 1562.60
EL. 1560.60
EL. 1557.41
EL. 1555.41
EL. 1552.22
EL. 1550.22
EL. 1547.03
EL. 1545.03
EL. 1541.84
EL. 1539.84
EL. 1536.65
EL. 1534.65
EL. 1531.46
EL. 1529.46
EL. 1526.27
EL. 1524.27
EL. 1521.08
EL. 1519.08
EL. 1515.89
EL. 1513.89
EL. 1510.70
EL. 1508.70
EL. 1505.51
EL. 1503.51
EL. 1500.32
EL. 1498.32
EL. 1495.13
EL. 1493.13
EL. 1489.94
EL. 1487.94
EL. 1484.75
EL. 1482.75
EL. 1479.56
EL. 1477.56
EL. 1474.37
EL. 1472.37
EL. 1469.18
EL. 1467.18
EL. 1463.99
EL. 1461.99
EL. 1458.80
EL. 1456.80
EL. 1453.61
EL. 1451.61
EL. 1448.42
EL. 1446.42
EL. 1443.23
EL. 1441.23
EL. 1438.04
EL. 1436.04
EL. 1432.85
EL. 1430.85
EL. 1427.66
EL. 1425.66
EL. 1422.47
EL. 1420.47
EL. 1417.28
EL. 1415.28
EL. 1412.09
EL. 1410.09
EL. 1406.90
EL. 1404.90
EL. 1401.71
EL. 1399.71
EL. 1396.52
EL. 1394.52
EL. 1391.33
EL. 1389.33
EL. 1386.14
EL. 1384.14
EL. 1380.95
EL. 1378.95
EL. 1375.76
EL. 1373.76
EL. 1370.57
EL. 1368.57
EL. 1365.38
EL. 1363.38
EL. 1360.19
EL. 1358.19
EL. 1355.00
EL. 1353.00
EL. 1349.81
EL. 1347.81
EL. 1344.62
EL. 1342.62
EL. 1339.43
EL. 1337.43
EL. 1334.24
EL. 1332.24
EL. 1329.05
EL. 1327.05
EL. 1323.86
EL. 1321.86
EL. 1318.67
EL. 1316.67
EL. 1313.48
EL. 1311.48
EL. 1308.29
EL. 1306.29
EL. 1303.10
EL. 1301.10
EL. 1297.91
EL. 1295.91
EL. 1292.72
EL. 1290.72
EL. 1287.53
EL. 1285.53
EL. 1282.34
EL. 1280.34
EL. 1277.15
EL. 1275.15
EL. 1271.96
EL. 1269.96
EL. 1266.77
EL. 1264.77
EL. 1261.58
EL. 1259.58
EL. 1256.39
EL. 1254.39
EL. 1251.20
EL. 1249.20
EL. 1246.01
EL. 1244.01
EL. 1240.82
EL. 1238.82
EL. 1235.63
EL. 1233.63
EL. 1230.44
EL. 1228.44
EL. 1225.25
EL. 1223.25
EL. 1220.06
EL. 1218.06
EL. 1214.87
EL. 1212.87
EL. 1209.68
EL. 1207.68
EL. 1204.49
EL. 1202.49
EL. 1199.30
EL. 1197.30
EL. 1194.11
EL. 1192.11
EL. 1188.92
EL. 1186.92
EL. 1183.73
EL. 1181.73
EL. 1178.54
EL. 1176.54
EL. 1173.35
EL. 1171.35
EL. 1168.16
EL. 1166.16
EL. 1162.97
EL. 1160.97
EL. 1157.78
EL. 1155.78
EL. 1152.59
EL. 1150.59
EL. 1147.40
EL. 1145.40
EL. 1142.21
EL. 1140.21
EL. 1137.02
EL. 1135.02
EL. 1131.83
EL. 1129.83
EL. 1126.64
EL. 1124.64
EL. 1121.45
EL. 1119.45
EL. 1116.26
EL. 1114.26
EL. 1111.07
EL. 1109.07
EL. 1105.88
EL. 1103.88
EL. 1100.69
EL. 1098.69
EL. 1095.50
EL. 1093.50
EL. 1090.31
EL. 1088.31
EL. 1085.12
EL. 1083.12
EL. 1079.93
EL. 1077.93
EL. 1074.74
EL. 1072.74
EL. 1069.55
EL. 1067.55
EL. 1064.36
EL. 1062.36
EL. 1059.17
EL. 1057.17
EL. 1053.98
EL. 1051.98
EL. 1048.79
EL. 1046.79
EL. 1043.60
EL. 1041.60
EL. 1038.41
EL. 1036.41
EL. 1033.22
EL. 1031.22
EL. 1028.03
EL. 1026.03
EL. 1022.84
EL. 1020.84
EL. 1017.65
EL. 1015.65
EL. 1012.46
EL. 1010.46
EL. 1007.27
EL. 1005.27
EL. 1002.08
EL. 1000.08
EL. 996.89
EL. 994.89
EL. 991.70
EL. 989.70
EL. 986.51
EL. 984.51
EL. 981.32
EL. 979.32
EL. 976.13
EL. 974.13
EL. 970.94
EL. 968.94
EL. 965.75
EL. 963.75
EL. 960.56
EL. 958.56
EL. 955.37
EL. 953.37
EL. 950.18
EL. 948.18
EL. 944.99
EL. 942.99
EL. 939.80
EL. 937.80
EL. 934.61
EL. 932.61
EL. 929.42
EL. 927.42
EL. 924.23
EL. 922.23
EL. 919.04
EL. 917.04
EL. 913.85
EL. 911.85
EL. 908.66
EL. 906.66
EL. 903.47
EL. 901.47
EL. 898.28
EL. 896.28
EL. 893.09
EL. 891.09
EL. 887.90
EL. 885.90
EL. 882.71
EL. 880.71
EL. 877.52
EL. 875.52
EL. 872.33
EL. 870.33
EL. 867.14
EL. 865.14
EL. 861.95
EL. 859.95
EL. 856.76
EL. 854.76
EL. 851.57
EL. 849.57
EL. 846.38
EL. 844.38
EL. 841.19
EL. 839.19
EL. 836.00
EL. 834.00
EL. 830.81
EL. 828.81
EL. 825.62
EL. 823.62
EL. 820.43
EL. 818.43
EL. 815.24
EL. 813.24
EL. 810.05
EL. 808.05
EL. 804.86
EL. 802.86
EL. 799.67
EL. 797.67
EL. 794.48
EL. 792.48
EL. 789.29
EL. 787.29
EL. 784.10
EL. 782.10
EL. 778.91
EL. 776.91
EL. 773.72
EL. 771.72
EL. 768.53
EL. 766.53
EL. 763.34
EL. 761.34
EL. 758.15
EL. 756.15
EL. 752.96
EL. 750.96
EL. 747.77
EL. 745.77
EL. 742.58
EL. 740.58
EL. 737.39
EL. 735.39
EL. 732.20
EL. 730.20
EL. 727.01
EL. 725.01
EL. 721.82
EL. 719.82
EL. 716.63
EL. 714.63
EL. 711.44
EL. 709.44
EL. 706.25
EL. 704.25
EL. 701.06
EL. 699.06
EL. 695.87
EL. 693.87
EL. 690.68
EL. 688.68
EL. 685.49
EL. 683.49
EL. 680.30
EL. 678.30
EL. 675.11
EL. 673.11
EL. 669.92
EL. 667.92
EL. 664.73
EL. 662.73
EL. 659.54
EL. 657.54
EL. 654.35
EL. 652.35
EL. 649.16
EL. 647.16
EL. 643.97
EL. 641.97
EL. 638.78
EL. 636.78
EL. 633.59
EL. 631.59
EL. 628.40
EL. 626.40
EL. 623.21
EL. 621.21
EL. 618.02
EL. 616.02
EL. 612.83
EL. 610.83
EL. 607.64
EL. 605.64
EL. 602.45
EL. 600.45
EL. 597.26
EL. 595.26
EL. 592.07
EL. 590.07
EL. 586.88
EL. 584.88
EL. 581.69
EL. 579.69
EL. 576.50
EL. 574.50
EL. 571.31
EL. 569.31
EL. 566.12
EL. 564.12
EL. 560.93
EL. 558.93
EL. 555.74
EL. 553.74
EL. 550.55
EL. 548.55
EL. 545.36
EL. 543.36
EL. 540.17
EL. 538.17
EL. 534.98
EL. 532.98
EL. 529.79
EL. 527.79
EL. 524.60
EL. 522.60
EL. 519.41
EL. 517.41
EL. 514.22
EL. 512.22
EL. 509.03
EL. 507.03
EL. 503.84
EL. 501.84
EL. 498.65
EL. 496.65
EL. 493.46
EL. 491.46
EL. 488.27
EL. 486.27
EL. 483.08
EL. 481.08
EL. 477.89
EL. 475.89
EL. 472.70
EL. 470.70
EL. 467.51
EL. 465.51
EL. 462.32
EL. 460.32
EL. 457.13
EL. 455.13
EL. 451.94
EL. 449.94
EL. 446.75
EL. 444.75
EL. 441.56
EL. 439.56
EL. 436.37
EL. 434.37
EL. 431.18
EL. 429.18
EL. 425.99
EL. 423.99
EL. 420.80
EL. 418.80
EL. 415.61
EL. 413.61
EL. 410.42
EL. 408.42
EL. 405.23
EL. 403.23
EL. 400.04
EL. 398.04
EL. 394.85
EL. 392.85
EL. 389.66
EL. 387.66
EL. 384.47
EL. 382.47
EL. 379.28
EL. 377.28
EL. 374.09
EL. 372.09
EL. 368.90
EL. 366.90
EL. 363.71
EL. 361.71
EL. 358.52
EL. 356.52
EL. 353.33
EL. 351.33
EL. 348.14
EL. 346.14
EL. 342.95
EL. 340.95
EL. 337.76
EL. 335.76
EL. 332.57
EL. 330.57
EL. 327.38
EL. 325.38
EL. 322.19
EL. 320.19
EL. 317.00
EL. 315.00
EL. 311.81
EL. 309.81
EL. 306.62
EL. 304.62
EL. 301.43
EL. 299.43
EL. 296.24
EL. 294.24
EL. 291.05
EL. 289.05
EL. 285.86
EL. 283.86
EL. 280.67
EL. 278.67
EL. 275.48
EL. 273.48
EL. 270.29
EL. 268.29
EL. 265.10
EL. 263.10
EL. 259.91
EL. 257.91
EL. 254.72
EL. 252.72
EL. 249.53
EL. 247.53
EL. 244.34
EL. 242.34
EL. 239.15
EL. 237.15
EL. 233.96
EL. 231.96
EL. 228.77
EL. 226.77
EL. 223.58
EL. 221.58
EL. 218.39
EL. 216.39
EL. 213.20
EL. 211.20
EL. 208.01
EL. 206.01
EL. 202.82
EL. 200.82
EL. 197.63
EL. 195.63
EL. 192.44
EL. 190.44
EL. 187.25
EL. 185.25
EL. 182.06
EL. 180.06
EL. 176.87
EL. 174.87
EL. 171.68
EL. 169.68
EL. 166.49
EL. 164.49
EL. 161.30
EL. 159.30
EL. 156.11
EL. 154.11
EL. 150.92
EL. 148.92
EL. 145.73
EL. 143.73
EL. 140.54
EL. 138.54
EL. 135.35
EL. 133.35
EL. 130.16
EL. 128.16
EL. 124.97
EL. 122.97
EL. 119.78
EL. 117.78
EL. 114.59
EL. 112.59
EL. 109.40
EL. 107.40
EL. 104.21
EL. 102.21
EL. 99.02
EL. 97.02
EL. 93.83
EL. 91.83
EL. 88.64
EL. 86.64
EL. 83.45
EL. 81.45
EL. 78.26
EL. 76.26
EL. 73.07
EL. 71.07
EL. 67.88
EL. 65.88
EL. 62.69
EL. 60.69
EL. 57.50
EL. 55.50
EL. 52.31
EL. 50.31
EL. 47.12
EL. 45.12
EL. 41.93
EL. 39.93
EL. 36.74
EL. 34.74
EL. 31.55
EL. 29.55
EL. 26.36
EL. 24.36
EL. 21.17
EL. 19.17
EL. 15.98
EL. 13.98
EL. 10.79
EL. 8.79
EL. 5.60
EL. 3.60
EL. 0.41
EL. -1.59
EL. -4.78
EL. -6.78
EL. -9.97
EL. -11.97
EL. -15.16
EL. -17.16
EL. -20.35
EL. -22.35
EL. -25.54
EL. -27.54
EL. -30.73
EL. -32.73
EL. -35.92
EL. -37.92
EL. -41.11
EL. -43.11
EL. -46.30
EL. -48.30
EL. -51.49
EL.



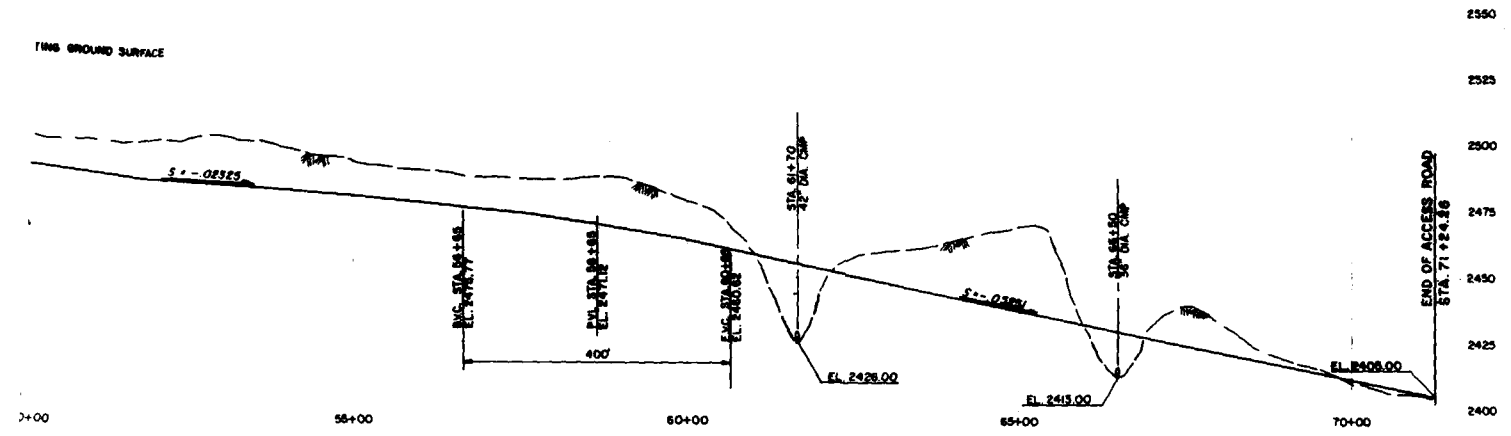
PROFILE
VERT. SCALE: 1 IN = 25 FT.
HORIZ SCALE: 1 IN = 100 FT.



ROAD ℄ CURVE DATA				
PI	Δ	R	T	L

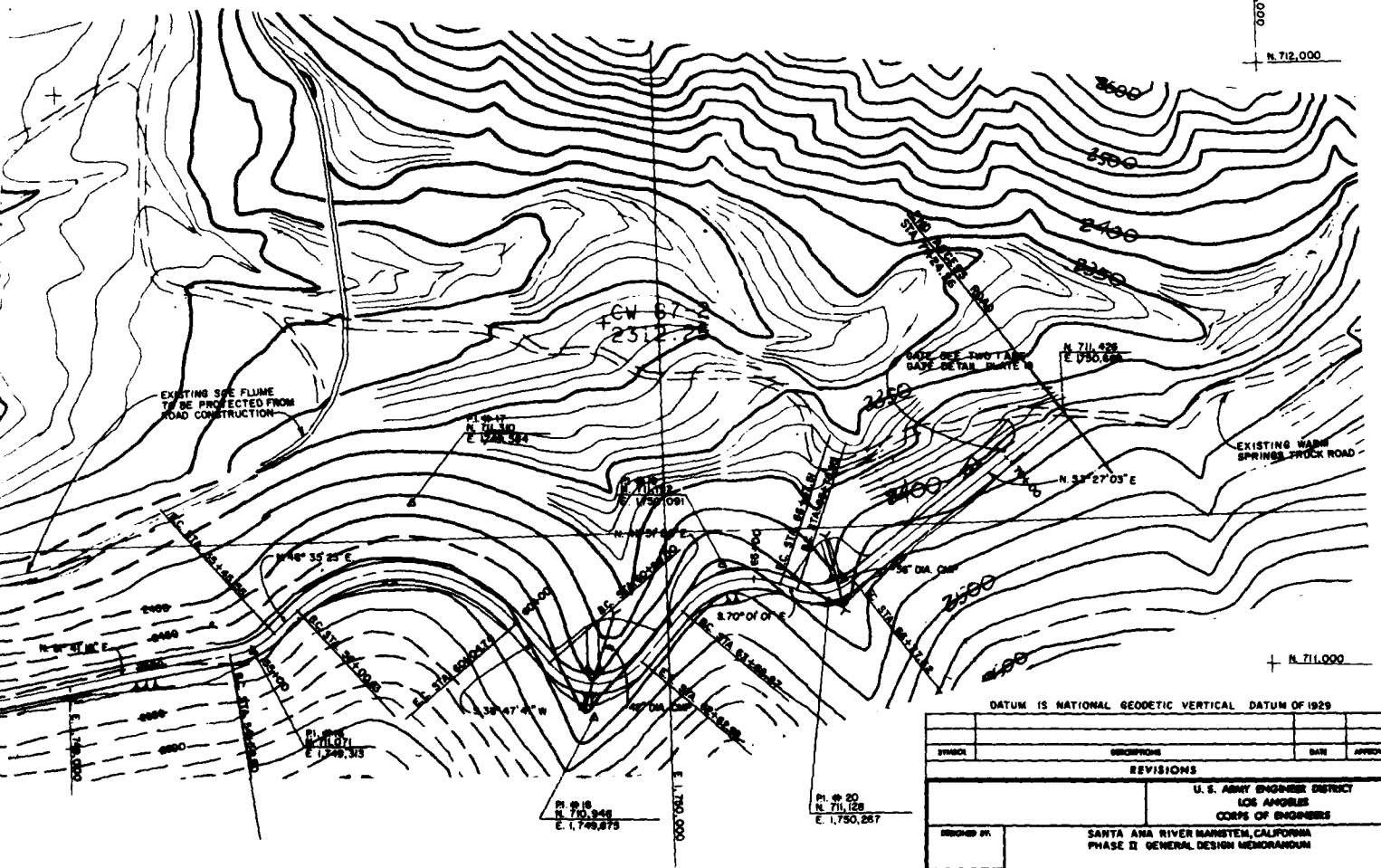
PLAN
SCALE 1 IN = 100 FT.

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING



PROFILE
VERT. SCALE: 1 IN. = 25 FT.
HORIZ. SCALE: 1 IN. = 100 FT.

FOR CUTS OVER 20 FT. HIGH SEE TYPICAL BENCH SECTION PLATE 16



PLAN
SCALE: 1 IN. = 100 FT.

SCALE: 1 IN. = 100 FT.
SCALE: 1 IN. = 25 FT.

DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

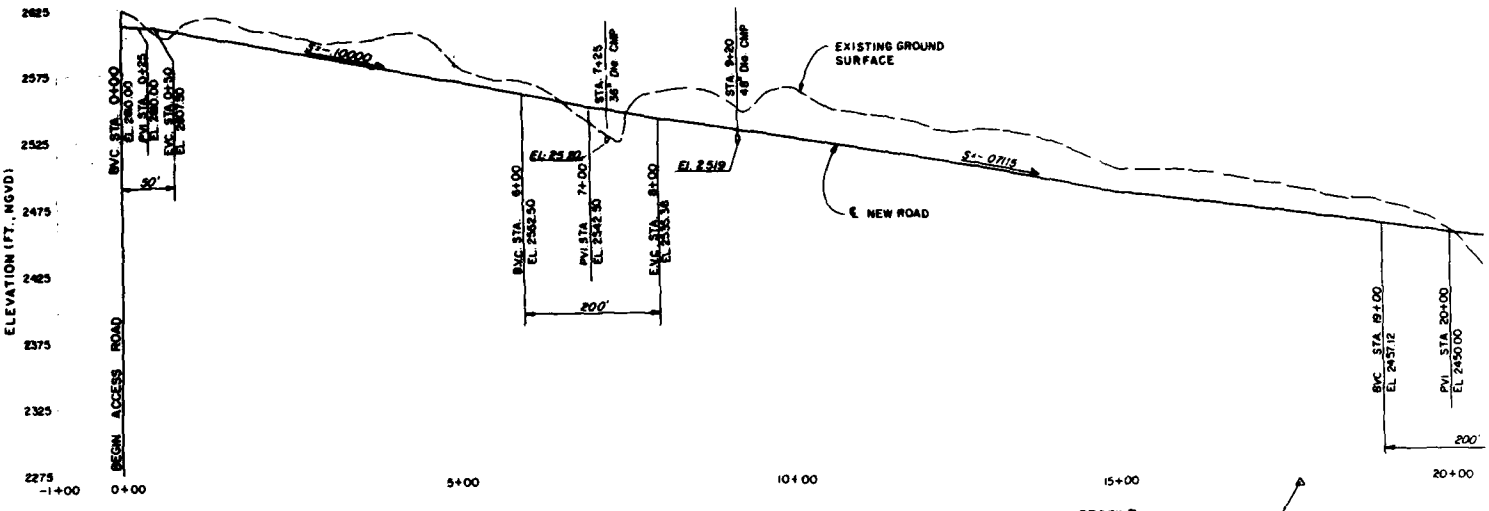
SPILLWAY TO WARM SPRINGS CANYON ACCESS ROAD PLAN, PROFILE AND SECTION
STA. 38+00 TO STA. 72+60.16

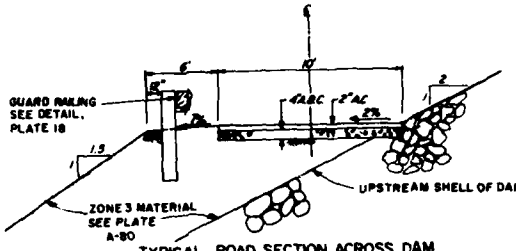
SUBMITTED BY: DATE APPROVED:

SPEC. NO. DACW09-__-__-____
DISTRICT FILE NO.

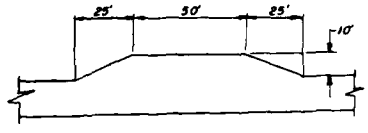
SHEET 2 OF 2

PLATE 80



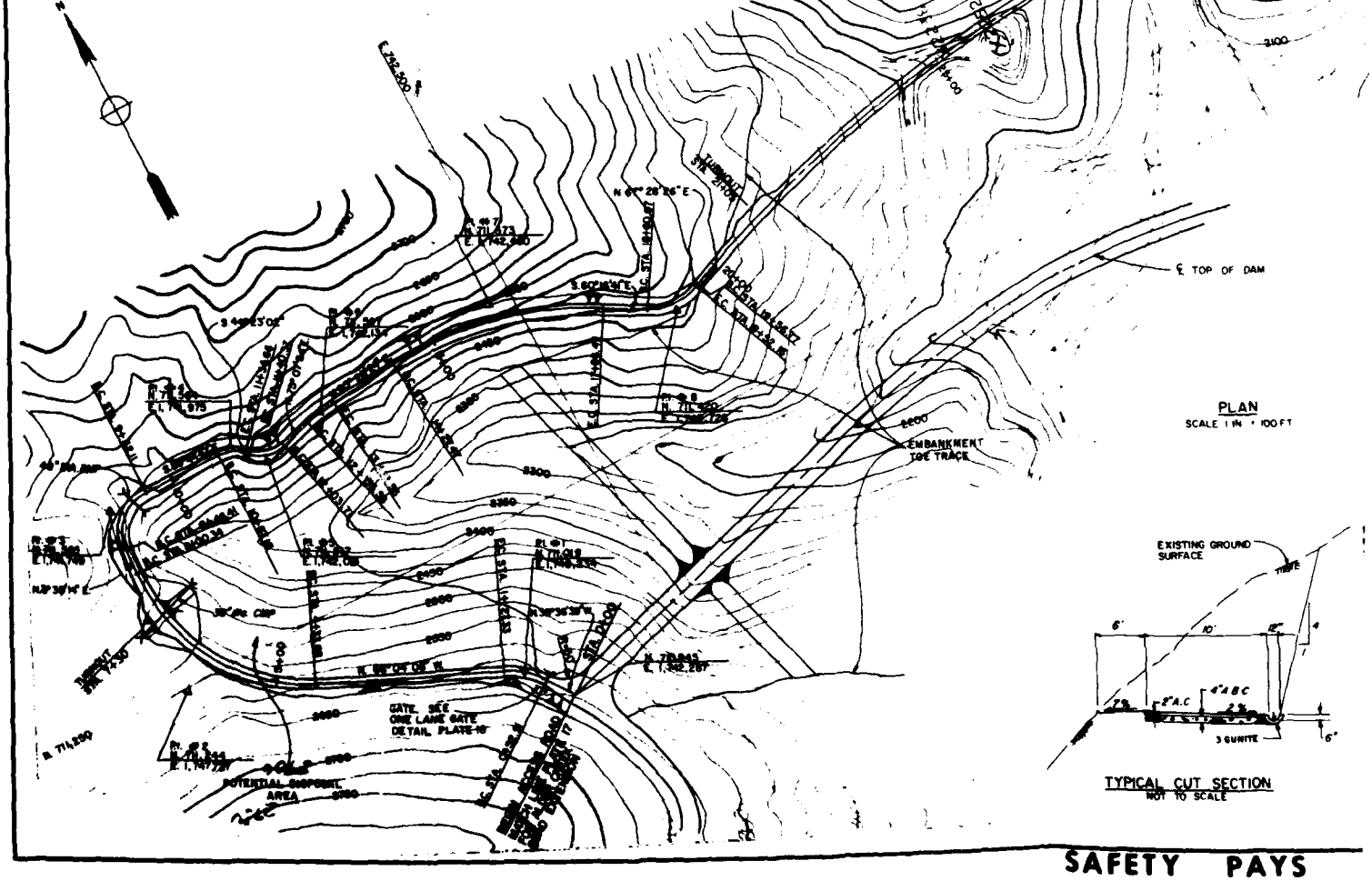


TYPICAL ROAD SECTION ACROSS DAM
NOT TO SCALE



TYPICAL TURNOUT PLAN DETAIL
NOT TO SCALE

PROFILE
HORIZ SCALE 1 IN = 100 FT
VERT SCALE 1 IN = 50 FT

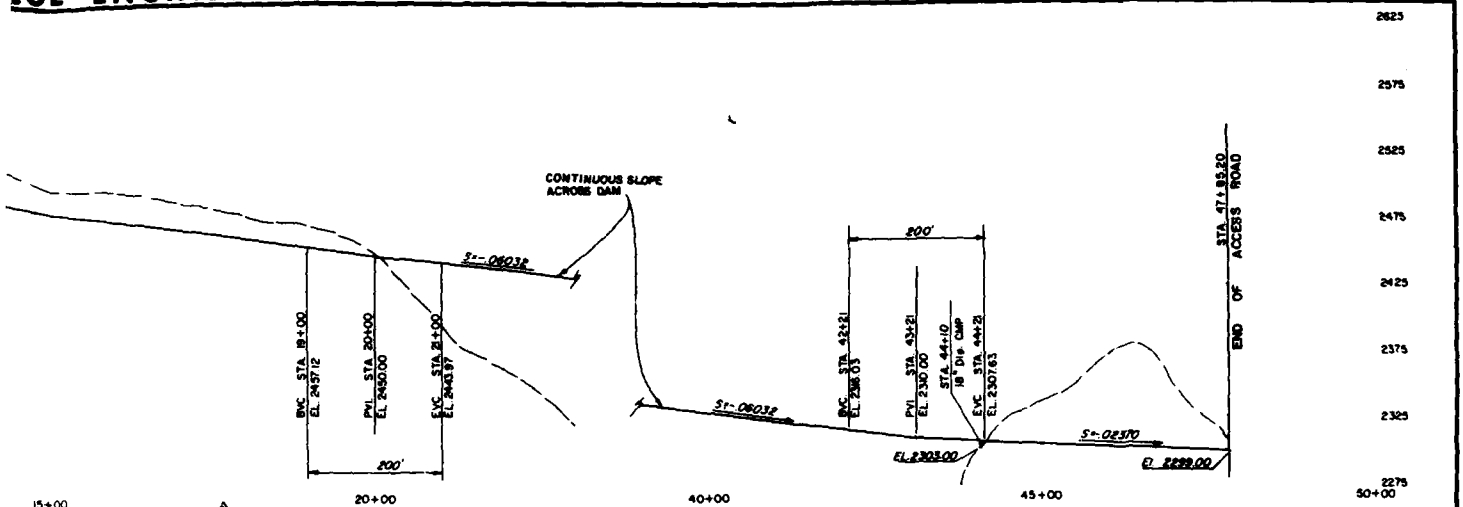


PLAN
SCALE 1 IN = 100 FT

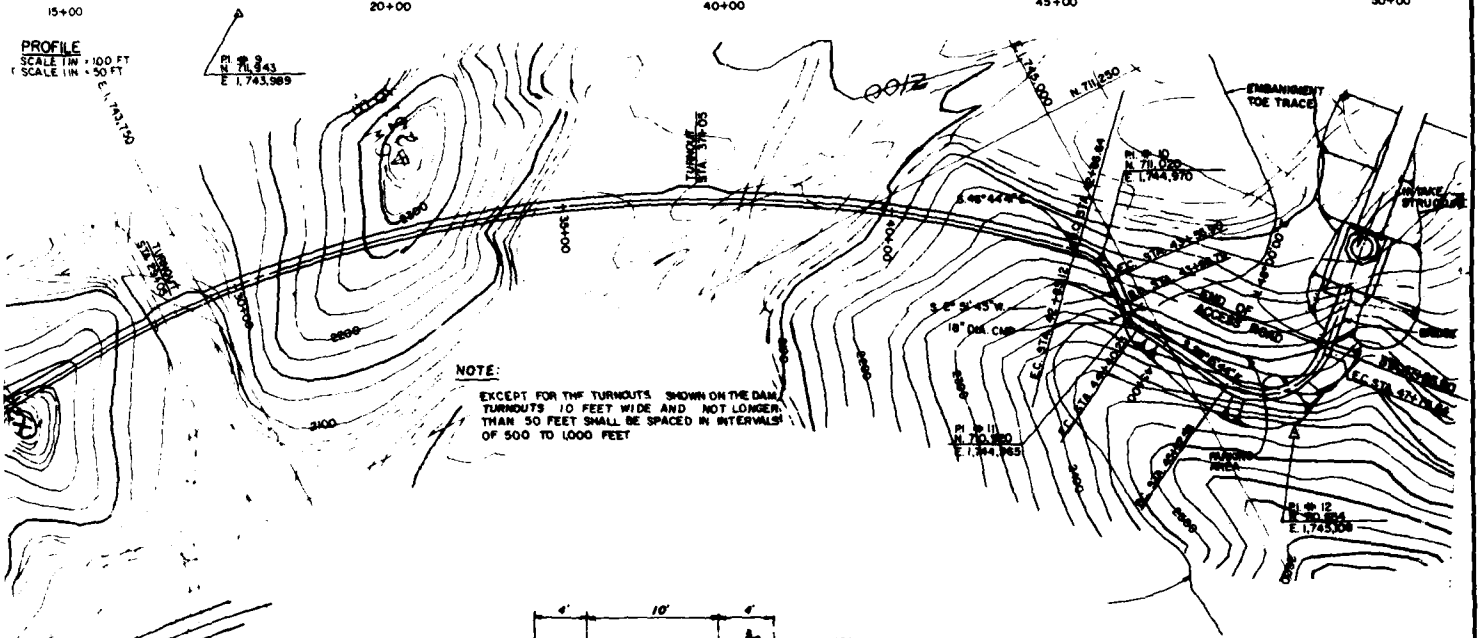
TYPICAL CUT SECTION
NOT TO SCALE

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

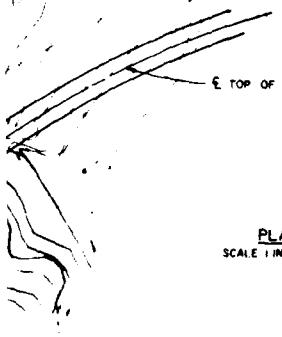
LUE ENGINEERING PAYS



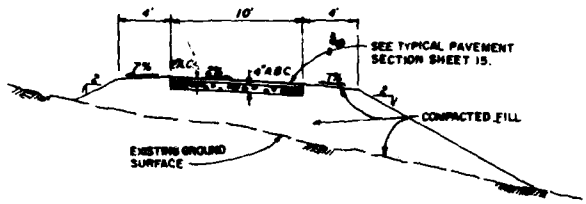
PROFILE
SCALE 1 IN = 100 FT
SCALE 1 IN = 50 FT



NOTE:
EXCEPT FOR THE TURNOUTS SHOWN ON THE DAM, TURNOUTS 10 FEET WIDE AND NOT LONGER THAN 50 FEET SHALL BE SPACED IN INTERVALS OF 500 TO 1,000 FEET

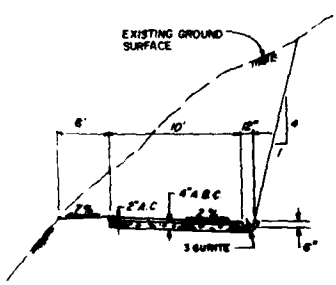


PLAN
SCALE 1 IN = 100 FT



TYPICAL FILL SECTION
NOT TO SCALE

ROAD ℄ CURVE DATA				
PI	Δ	R	T	L
1	30°27'39"	140'	38.34'	74.83'
2	98°59'23"	500'	210.08'	386.85'
3	88°19'36"	70'	65.88'	108.71'
4	19°21'45"	60'	25.08'	47.50'
5	60°29'04"	60'	34.98'	63.34'
6	17°18'30"	500'	27.06'	53.97'
7	34°16'54"	800'	185.05'	388.07'
8	56°14'53"	100'	49.64'	91.19'
9	69°46'53"	2025'	(311.81'	2339.35'
10	49°39'26"	80'	36.97'	69.35'
11	31°05'58"	80'	22.88'	43.42'
12	103°44'58"	100'	127.42'	181.08'



TYPICAL CUT SECTION
NOT TO SCALE

GUARDRAIL STATIONING	
LEFT SIDE OF ROAD	RIGHT SIDE OF ROAD
6+40 TO 7+70	0+25 TO 1+27
19+52 TO 47+85	4+34 TO 9+52
	10+87 TO 13+13
	14+27 TO 19+52

GUIDEPOST STATIONING	
LEFT SIDE OF ROAD	RIGHT SIDE OF ROAD
	1+27 TO 4+34
	9+52 TO 10+87
	13+13 TO 14+27

SCALE: 1 IN. = 100 FT.
100 0 100 200 300
SCALE: 1 IN. = 50 FT.
50 0 50 100 150

DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY: SPA
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM
TOP OF DAM TO INTAKE STRUCTURE
ACCESS ROAD PLAN, PROFILE, SECTIONS AND DETAIL

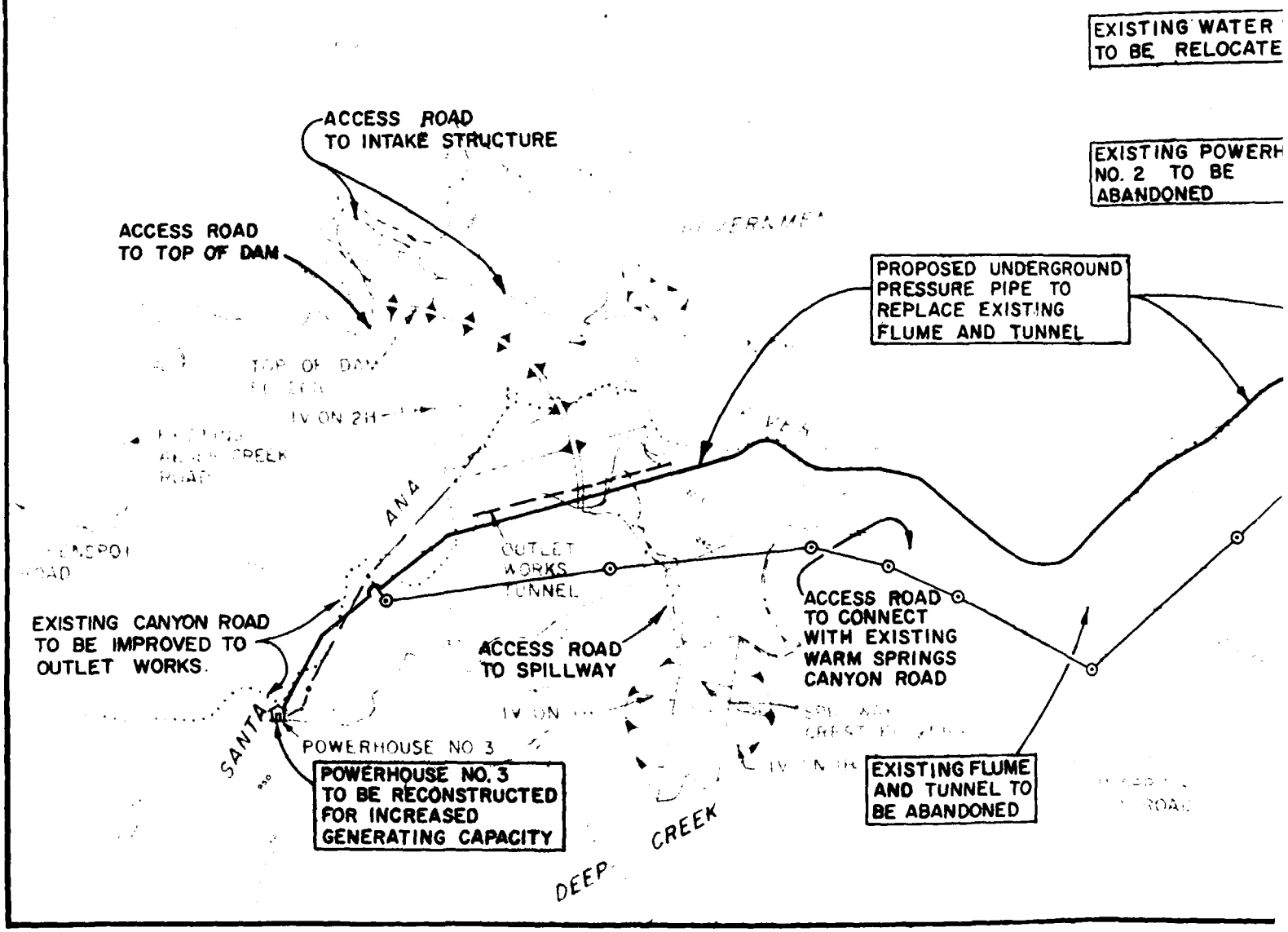
SUBMITTED BY:
DATE APPROVED:
CHIEF, ENGINEERING DIVISION
SPEC. NO. DACW09-____-B-____
DISTRICT FILE NO.
SHEET 1 OF 1

PLATE 21

SAFETY PAYS

SOURCE OF INFORMATION

CORPS OF ENGINEERS TOPOGRAPHY FROM AERIAL PHOTOGRAPHY FLOWN 24 JANUARY 1987.



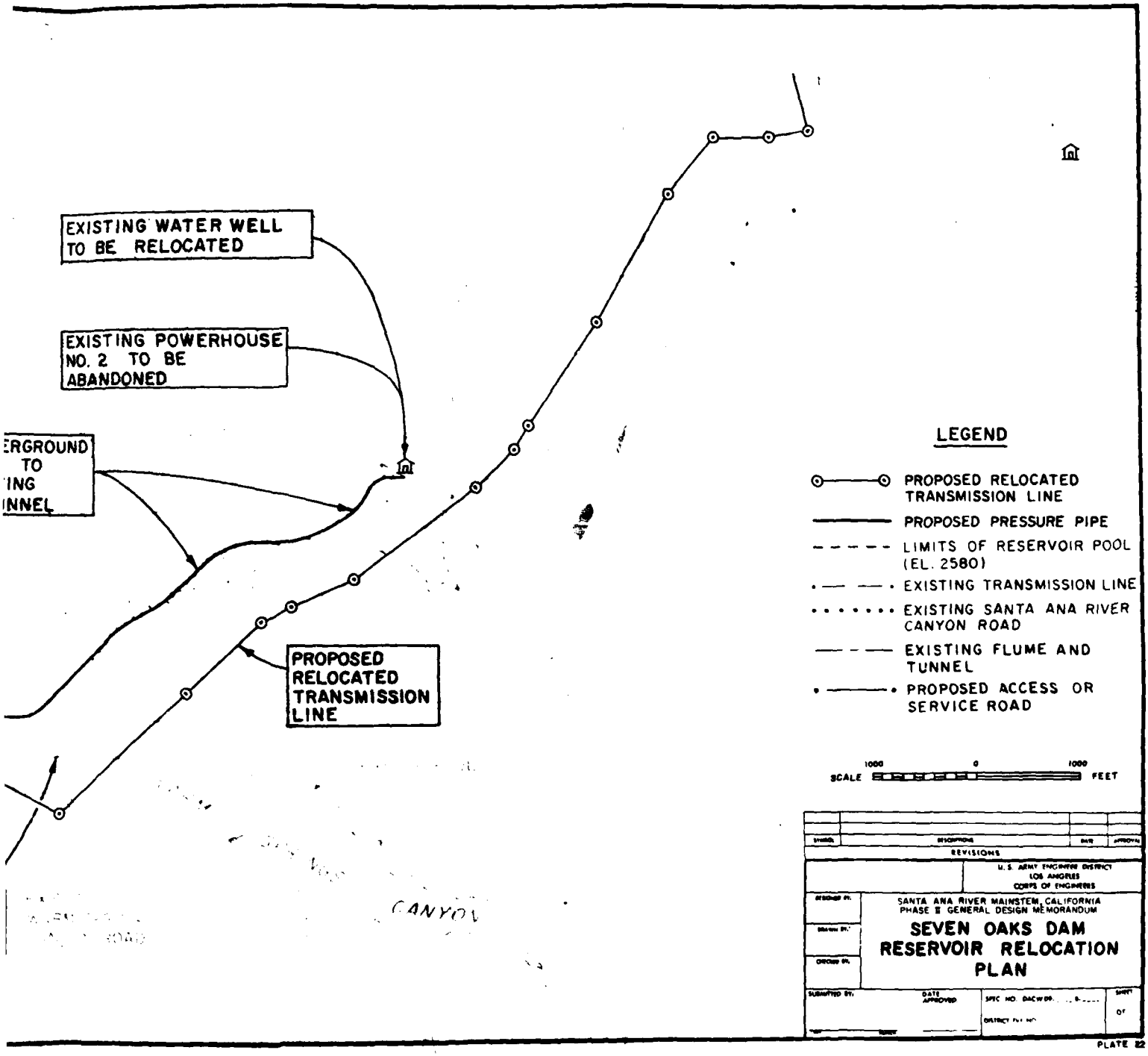
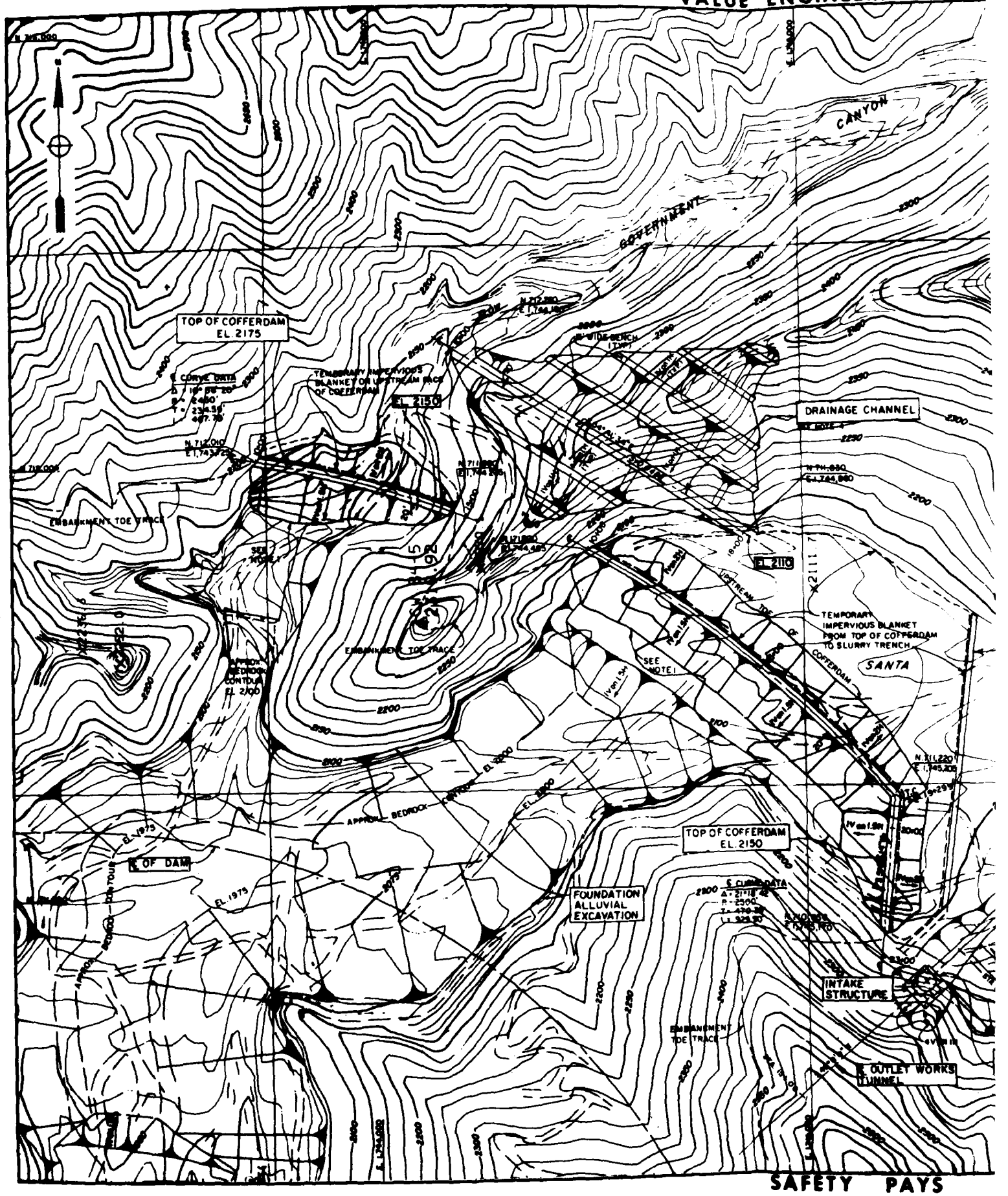
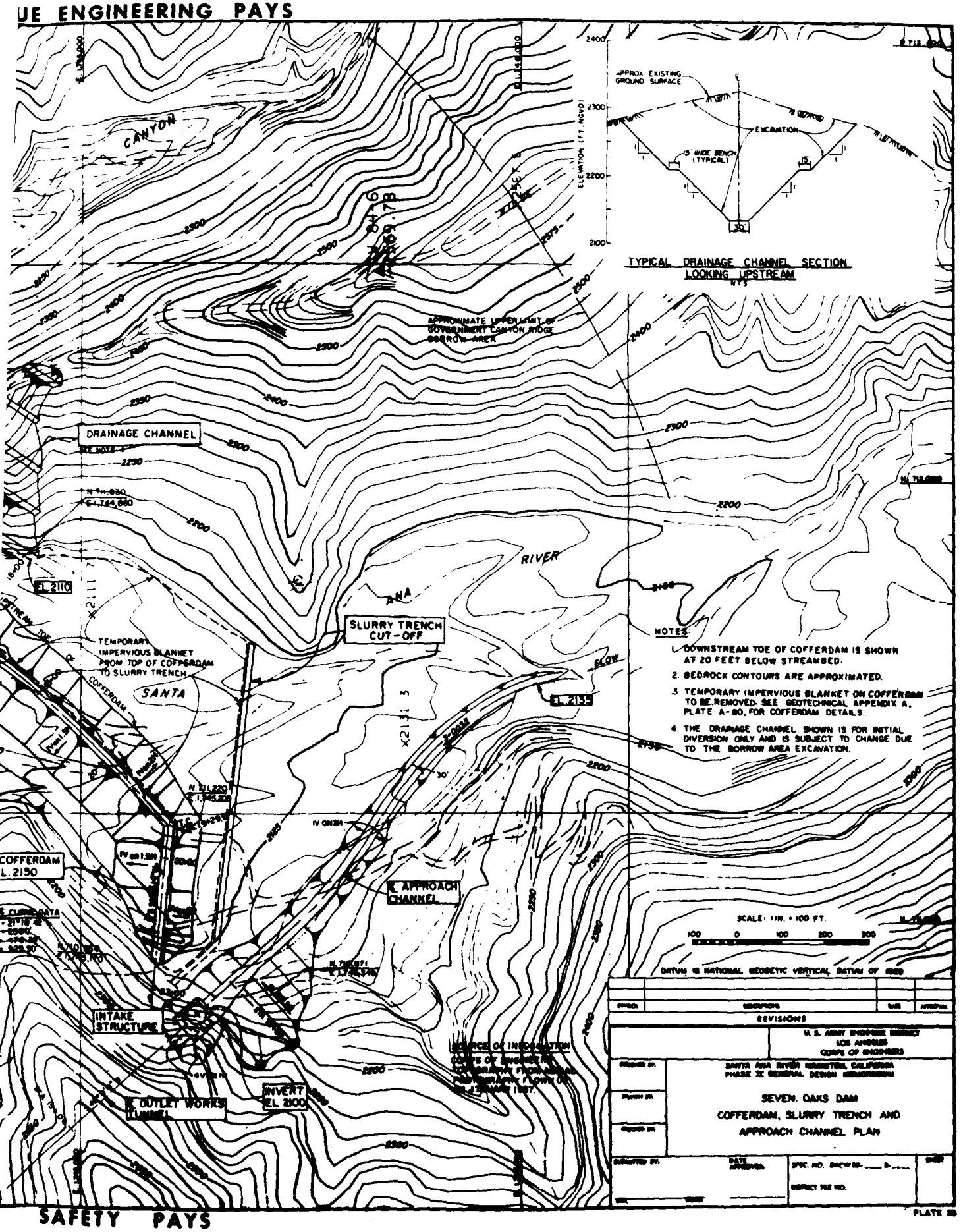

EXISTING WATER WELL TO BE RELOCATED
EXISTING POWERHOUSE NO. 2 TO BE ABANDONED
ERGROUND TO ING INNEL
PROPOSED RELOCATED TRANSMISSION LINE
CANYON
LEGEND
PROPOSED RELOCATED TRANSMISSION LINE
PROPOSED PRESSURE PIPE
LIMITS OF RESERVOIR POOL (EL. 2580)
EXISTING TRANSMISSION LINE
EXISTING SANTA ANA RIVER CANYON ROAD
EXISTING FLUME AND TUNNEL
PROPOSED ACCESS OR SERVICE ROAD
1000 0 1000
SCALE FEET
REVISIONS
U. S. ARMY ENGINEER DISTRICT LOS ANGELES CORPS OF ENGINEERS
SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM
SEVEN OAKS DAM
RESERVOIR RELOCATION
PLAN

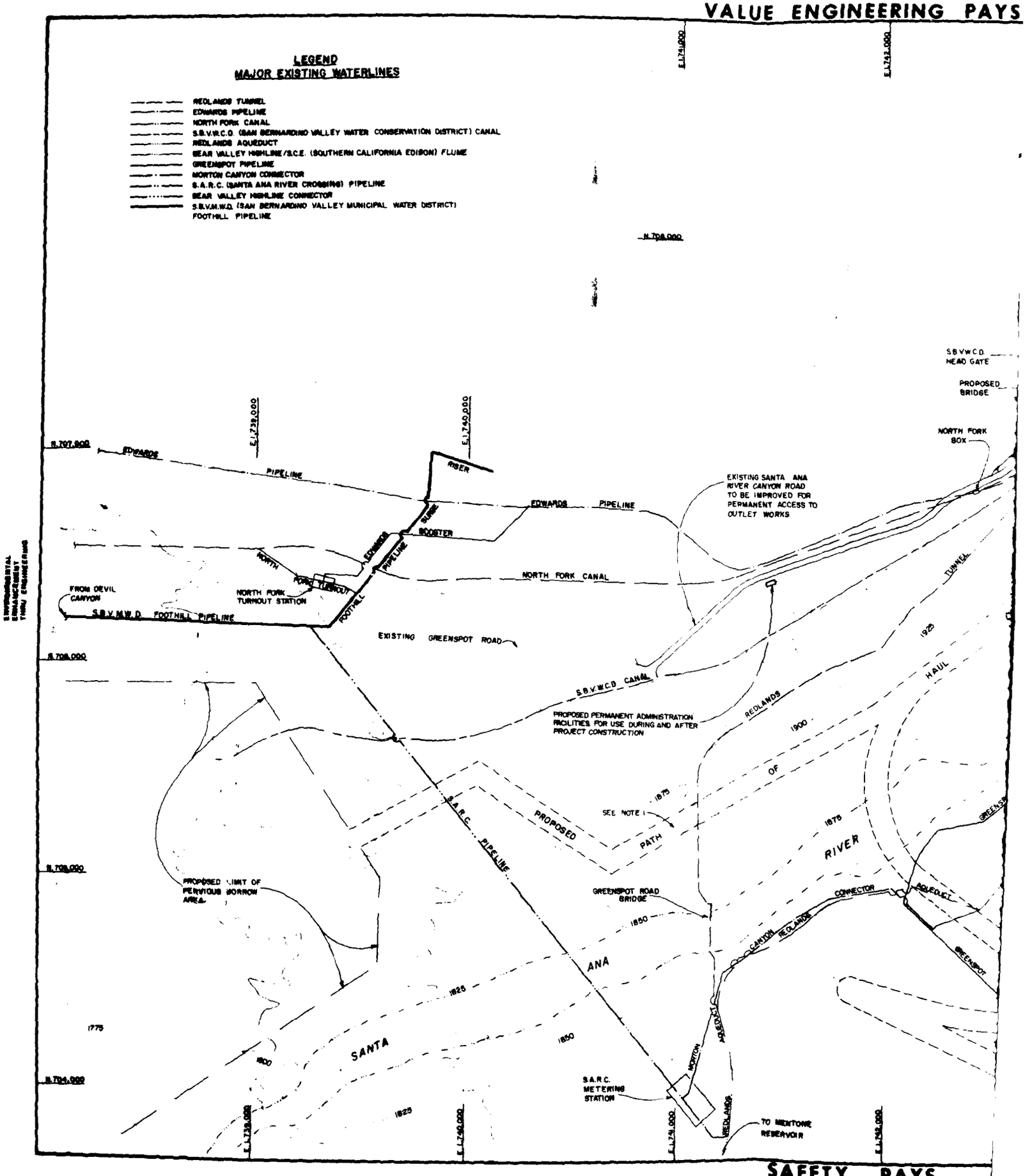
PLATE 22


TOP OF COFFERDAM
EL. 2175
TEMPORARY IMPERVIOUS BLANKET ON UPSTREAM FACE OF COFFERDAM
EL. 2150
DRAINAGE CHANNEL
EMBANKMENT TOE TRACE
EL. 2110
TEMPORARY IMPERVIOUS BLANKET FROM TOP OF COFFERDAM TO SLURRY TRENCH
SEE NOTE 1
APPROX. BEDROCK CONTOUR EL. 2100
TOP OF COFFERDAM
EL. 2150
FOUNDATION ALLUVIAL EXCAVATION
INTAKE STRUCTURE
EMBANKMENT TOE TRACE
OUTLET WORKS TUNNEL
CANYON
GOVERNMENT
SANTA
EL. 1975
APPROX. BEDROCK CONTOUR
EL. 2000
APPROX. BEDROCK CONTOUR
EL. 1975
E. OF DAM

UE ENGINEERING PAYS

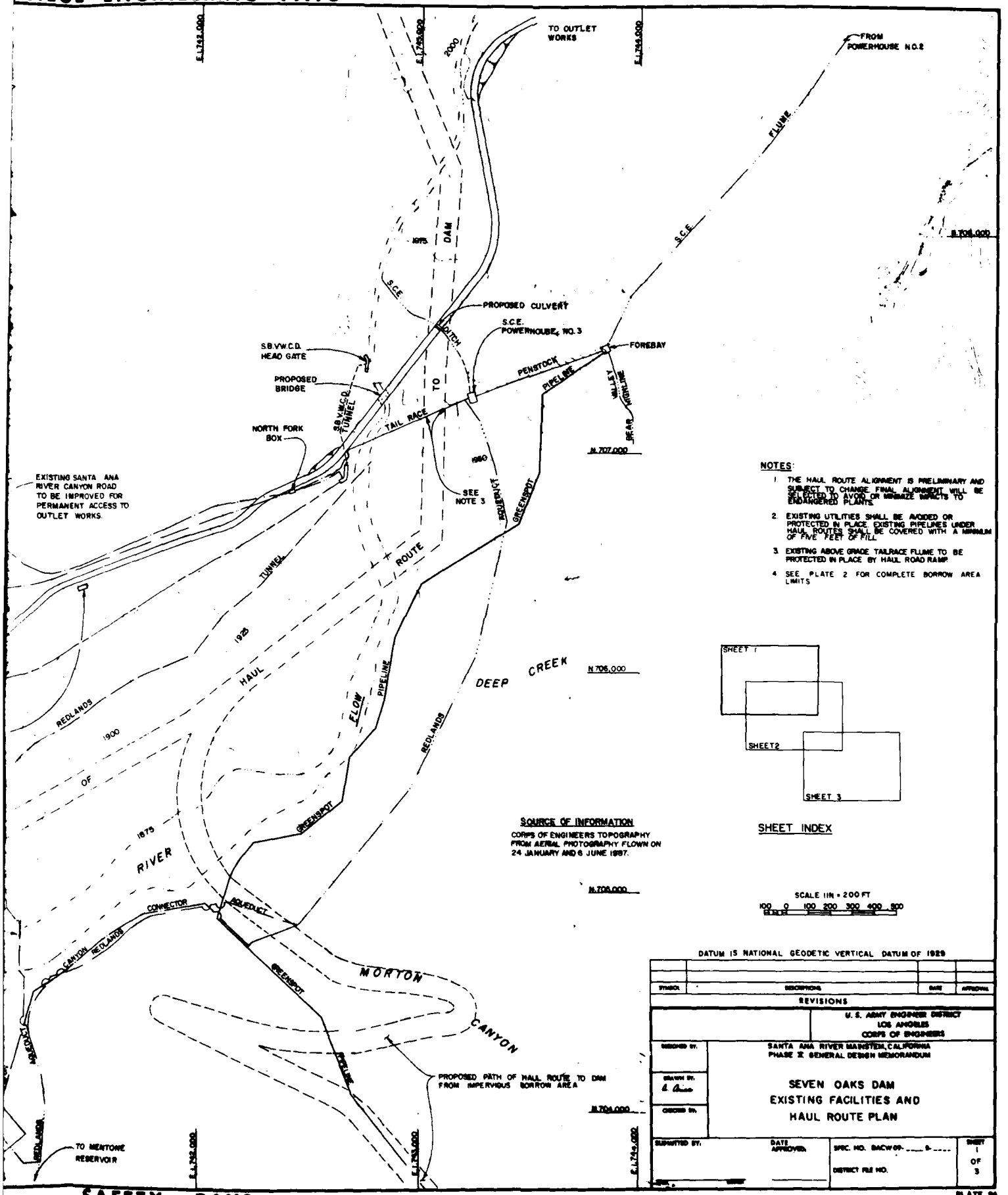


SAFETY PAYS



ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

VALUE ENGINEERING PAYS



NOTES:

1. THE HAUL ROUTE ALIGNMENT IS PRELIMINARY AND SUBJECT TO CHANGE. FINAL ALIGNMENT WILL BE SELECTED TO AVOID OR MINIMIZE IMPACTS TO ENDANGERED PLANTS.
2. EXISTING UTILITIES SHALL BE AVOIDED OR PROTECTED IN PLACE. EXISTING PIPELINES UNDER HAUL ROUTES SHALL BE COVERED WITH A MINIMUM OF FIVE FEET OF FILL.
3. EXISTING ABOVE GRADE TAILRACE FLUME TO BE PROTECTED IN PLACE BY HAUL ROAD RAMP.
4. SEE PLATE 2 FOR COMPLETE BORROW AREA LIMITS

SHEET INDEX

SOURCE OF INFORMATION

CORPS OF ENGINEERS TOPOGRAPHY FROM AERIAL PHOTOGRAPHY FLOWN ON 24 JANUARY AND 6 JUNE 1987.

SCALE 1IN = 200 FT

100 0 100 200 300 400 500

DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

SYMBOL	DESCRIPTION	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY: L. Cuevas
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

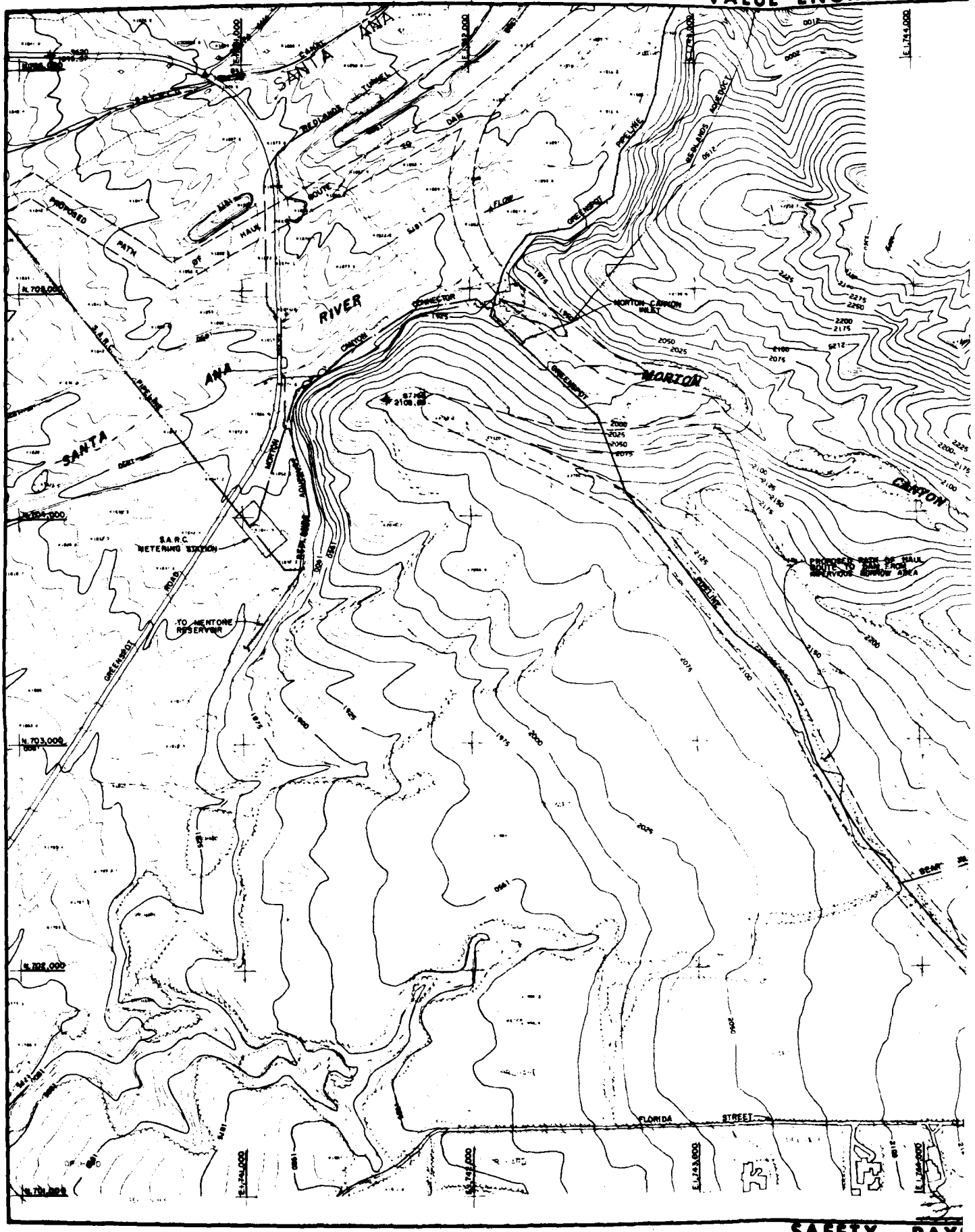
SEVEN OAKS DAM
EXISTING FACILITIES AND HAUL ROUTE PLAN

SUBMITTED BY:
DATE APPROVED:
SPEC. NO. DACW09-___-___
DISTRICT FILE NO.
SHEET 1 OF 3

PLATE 84

SAFETY PAYS

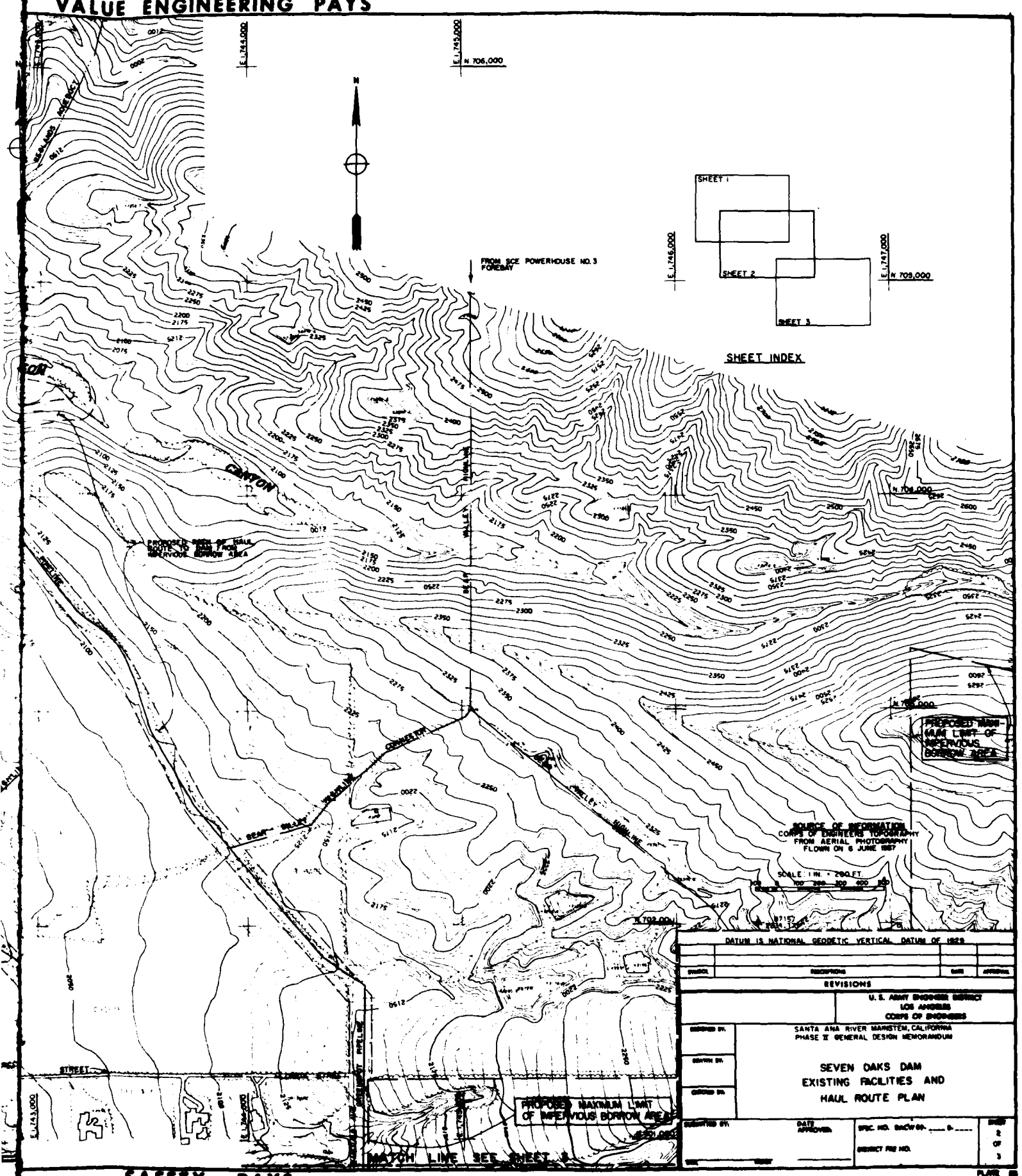
VALUE ENGINEERING



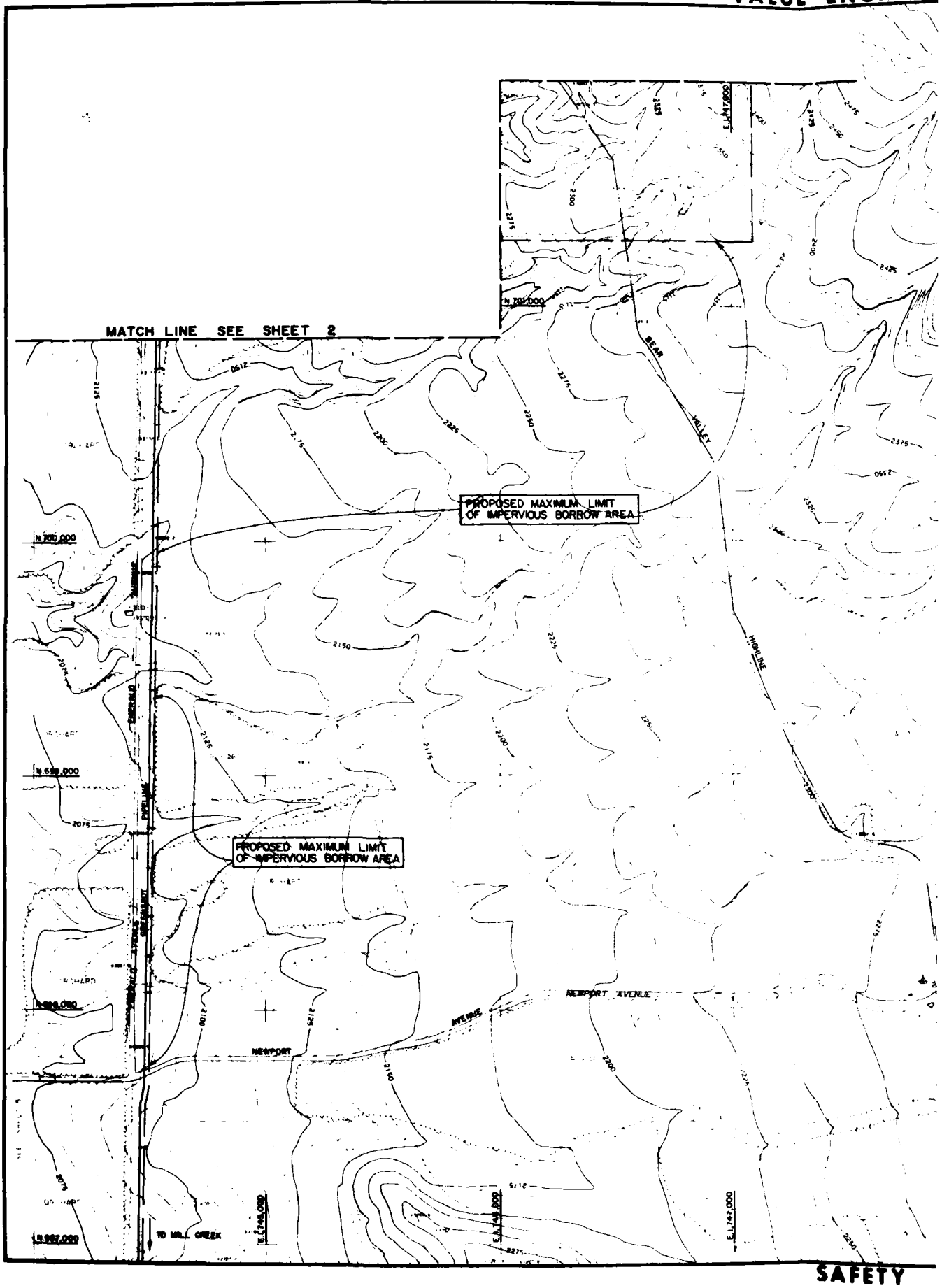
ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

SAFETY PAY

VALUE ENGINEERING PAYS

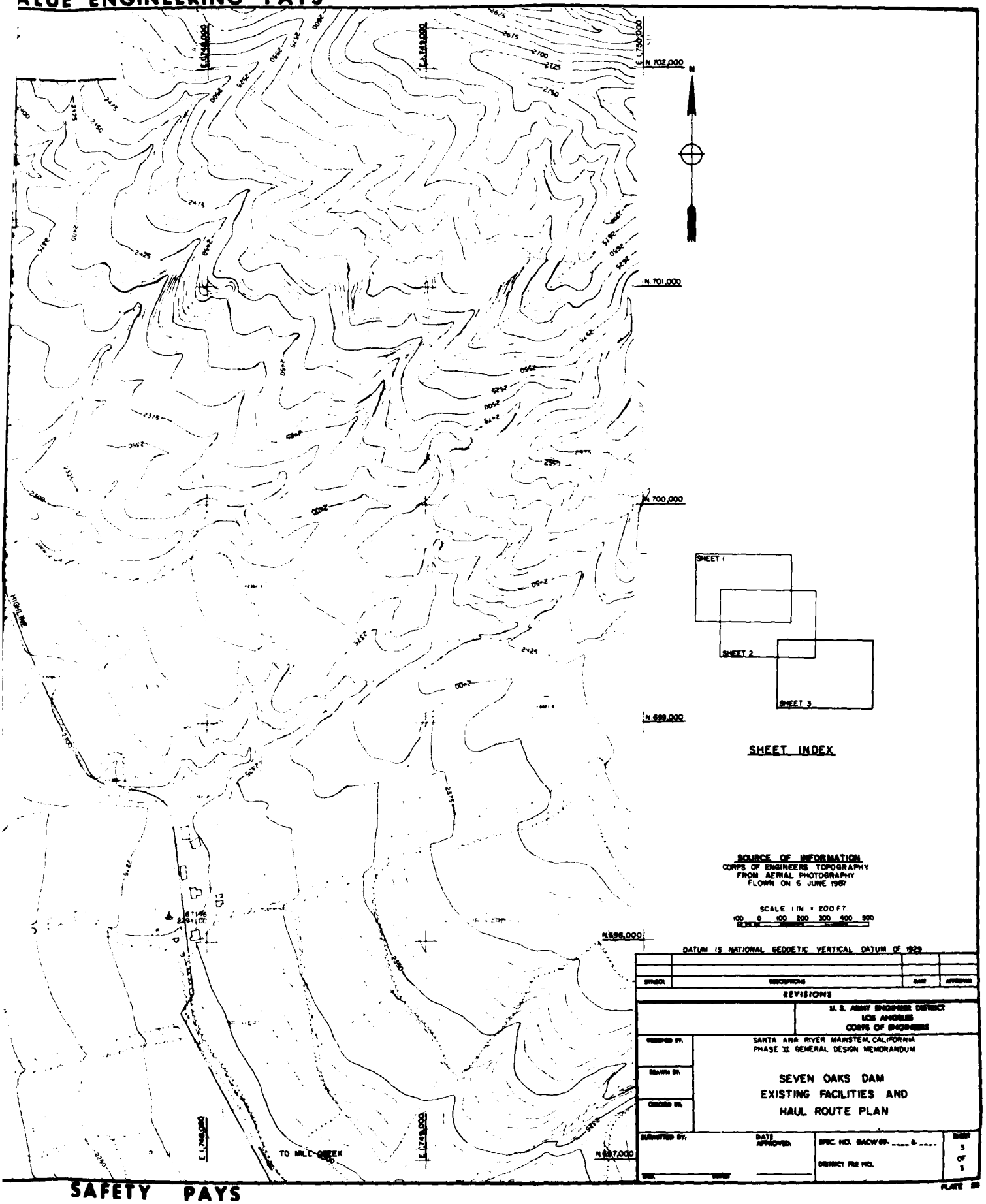


SAFETY PAYS



ENVIRONMENTAL
ENHANCEMENT
THRU ENGINEERING

ALUE ENGINEERING PAYS



SHEET INDEX

SOURCE OF INFORMATION
CORPS OF ENGINEERS TOPOGRAPHY
FROM AERIAL PHOTOGRAPHY
FLOWN ON 6 JUNE 1967

SCALE 1 IN = 200 FT
100 0 100 200 300 400 500

DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY
DRAWN BY
CHECKED BY

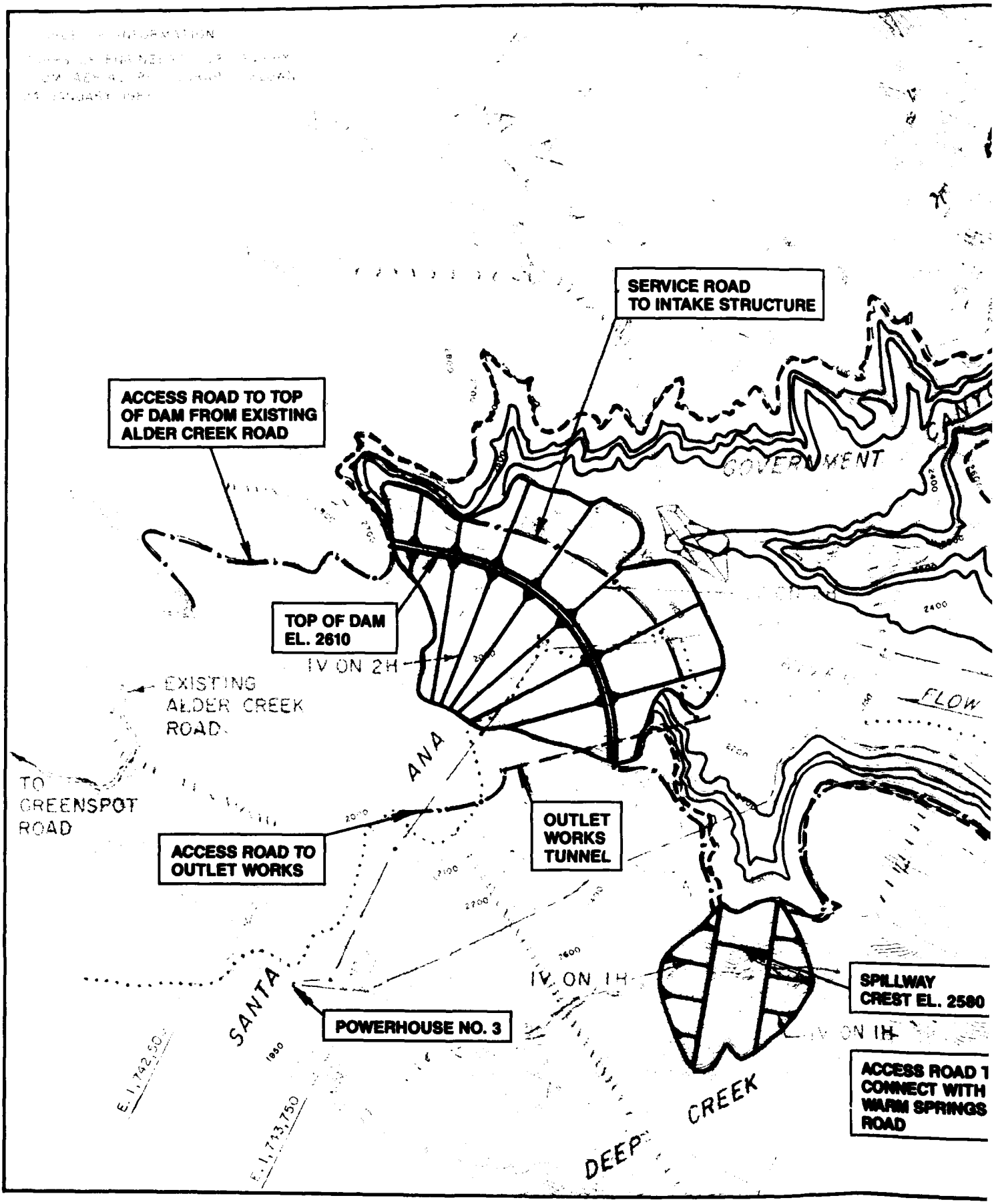
SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

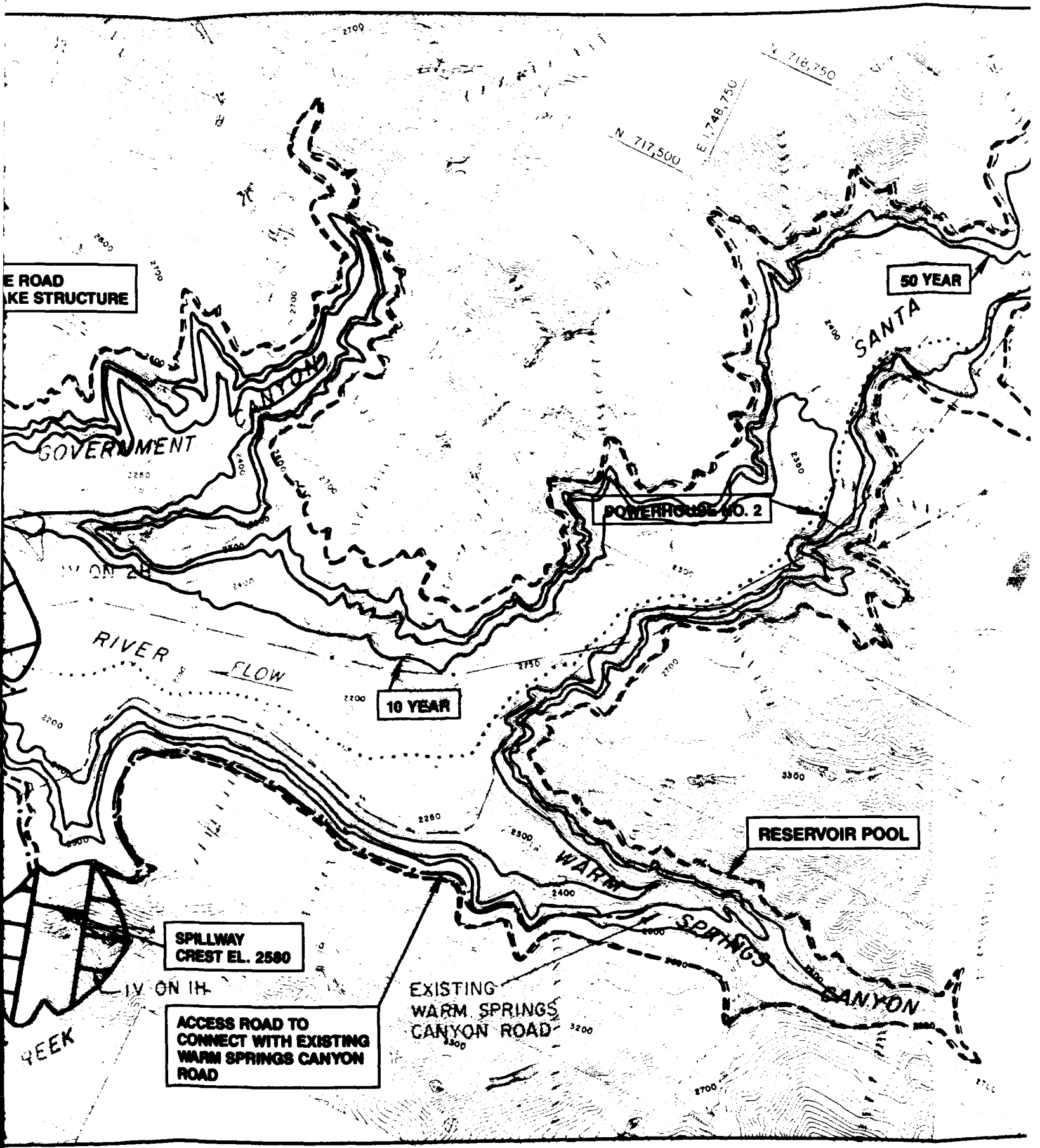
SEVEN OAKS DAM
EXISTING FACILITIES AND
HAUL ROUTE PLAN

SUBMITTED BY | DATE APPROVED | SPEC. NO. SACW09-____-S-____ | DISTRICT FILE NO. | SHEET 3 OF 3

PLATE 20

SAFETY PAYS


SERVICE ROAD TO INTAKE STRUCTURE
ACCESS ROAD TO TOP OF DAM FROM EXISTING ALDER CREEK ROAD
TOP OF DAM EL. 2610
1V ON 2H
EXISTING ALDER CREEK ROAD
TO GREENSPOT ROAD
ACCESS ROAD TO OUTLET WORKS
OUTLET WORKS TUNNEL
POWERHOUSE NO. 3
SPILLWAY CREST EL. 2580
ACCESS ROAD T CONNECT WITH WARM SPRINGS ROAD
SANTA ANA
DEEP CREEK
FLOW
GOVERNMENT
E. 1,742,50
E. 1,743,750


E ROAD
KE STRUCTURE
50 YEAR
SANTA
GOVERNMENT
CANYON
POWERHOUSE NO. 2
IV ON 2R
RIVER
FLOW
10 YEAR
RESERVOIR POOL
WARM
SPRINGS
CANYON
SPILLWAY
CREST EL. 2580
IV ON IH
ACCESS ROAD TO
CONNECT WITH EXISTING
WARM SPRINGS CANYON
ROAD
EXISTING
WARM SPRINGS
CANYON ROAD
REEK
N 717,500
E 1,748,750
N 718,750

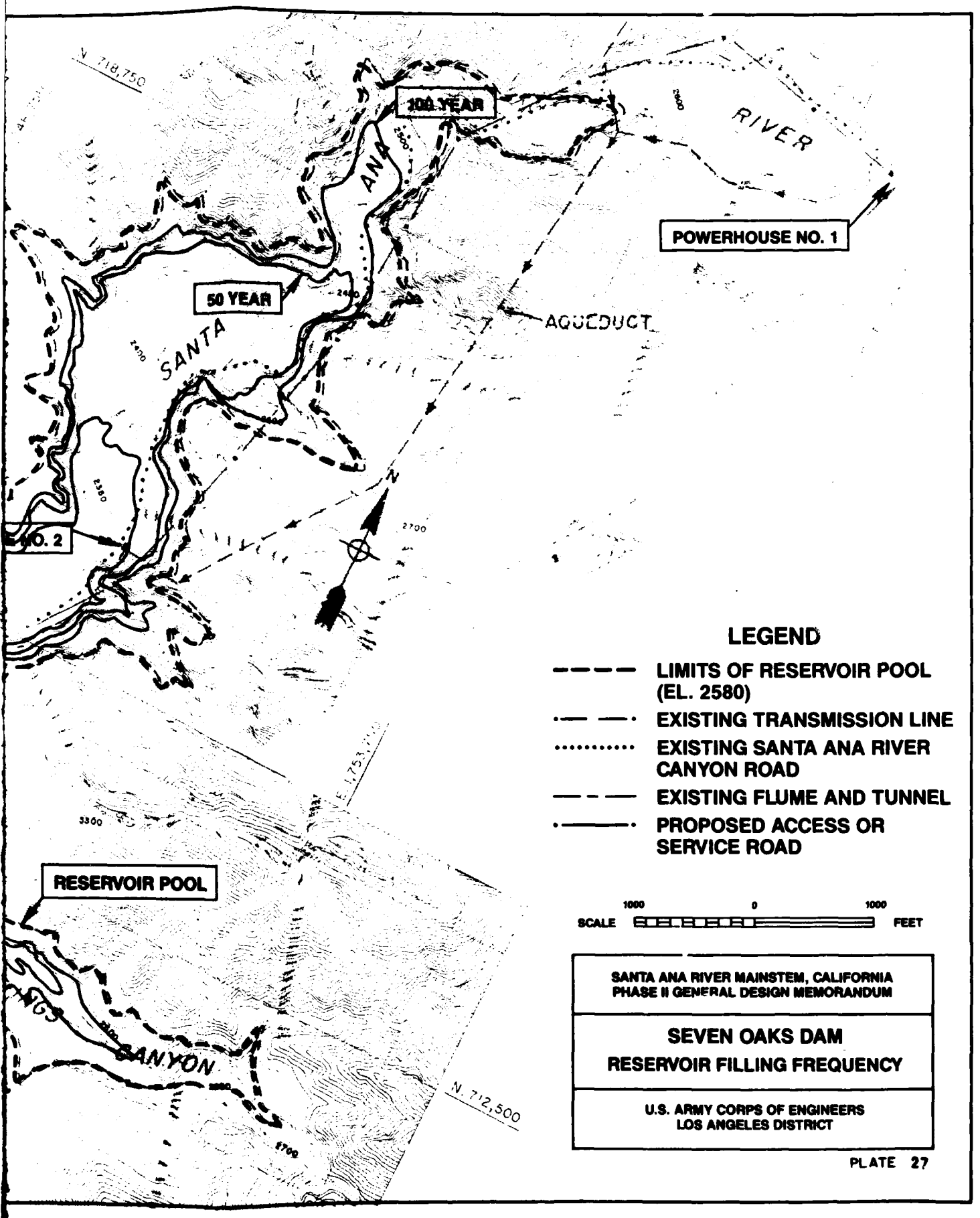
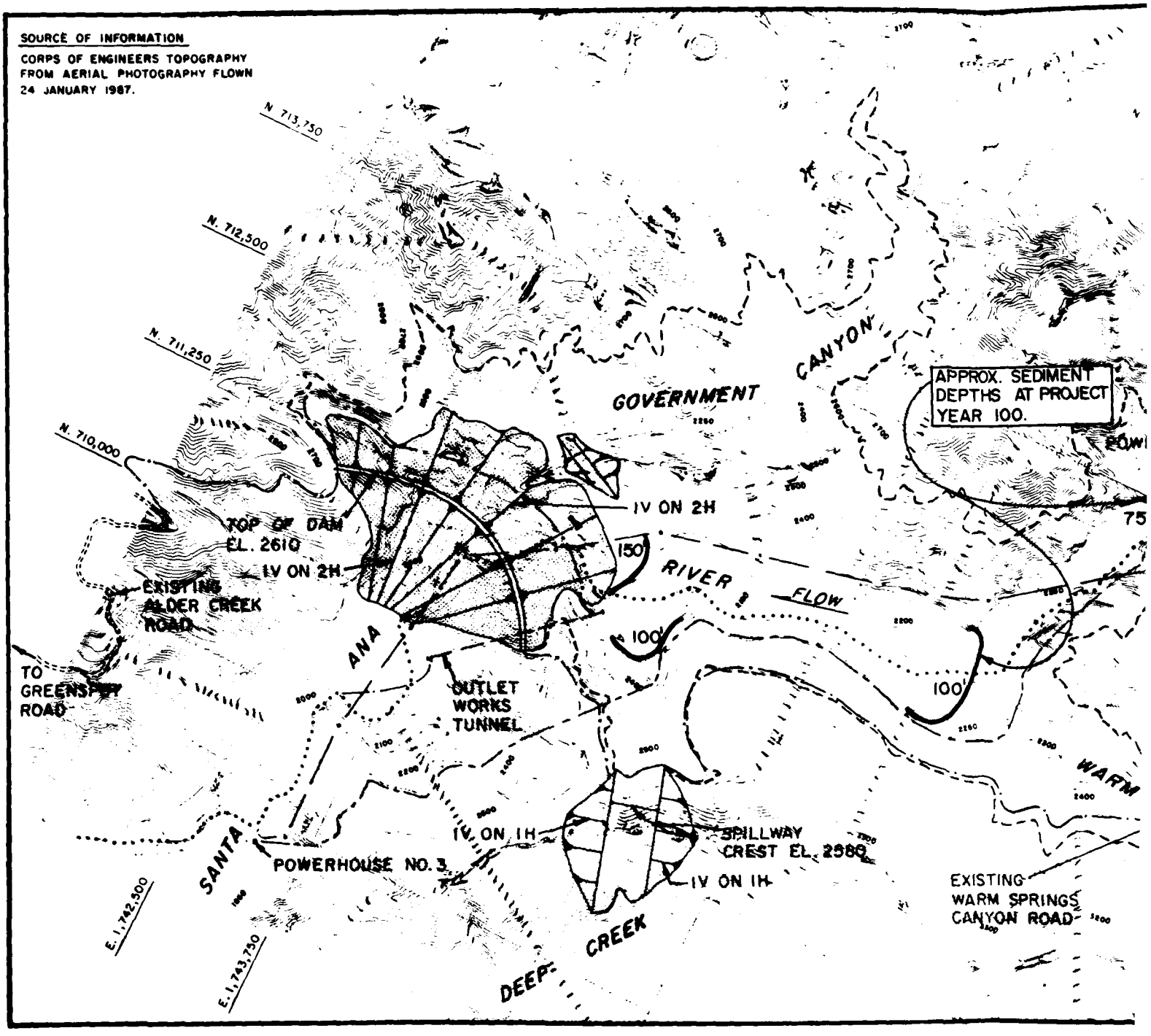

N. 718,750
100 YEAR
SANTA ANA
RIVER
POWERHOUSE NO. 1
50 YEAR
SANTA
AQUEDUCT
NO. 2
RESERVOIR POOL
CANYON
N. 712,500
LEGEND
LIMITS OF RESERVOIR POOL (EL. 2580)
EXISTING TRANSMISSION LINE
EXISTING SANTA ANA RIVER CANYON ROAD
EXISTING FLUME AND TUNNEL
PROPOSED ACCESS OR SERVICE ROAD
1000
0
1000
SCALE
FEET
SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM
SEVEN OAKS DAM
RESERVOIR FILLING FREQUENCY
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

PLATE 27


SOURCE OF INFORMATION
CORPS OF ENGINEERS TOPOGRAPHY
FROM AERIAL PHOTOGRAPHY FLOWN
24 JANUARY 1987.
N. 713,750
N. 712,500
N. 711,250
N. 710,000
APPROX. SEDIMENT DEPTHS AT PROJECT YEAR 100.
GOVERNMENT CANYON
1V ON 2H
TOP OF DAM EL. 2610
1V ON 2H
EXISTING ALDER CREEK ROAD
TO GREENSPOT ROAD
150'
100'
100'
RIVER FLOW
SANTA ANA
OUTLET WORKS TUNNEL
POWERHOUSE NO. 3
1V ON 1H
SPILLWAY CREST EL. 2580
1V ON 1H
EXISTING WARM SPRINGS CANYON ROAD
DEEP CREEK
E. 1,742,500
E. 1,743,750

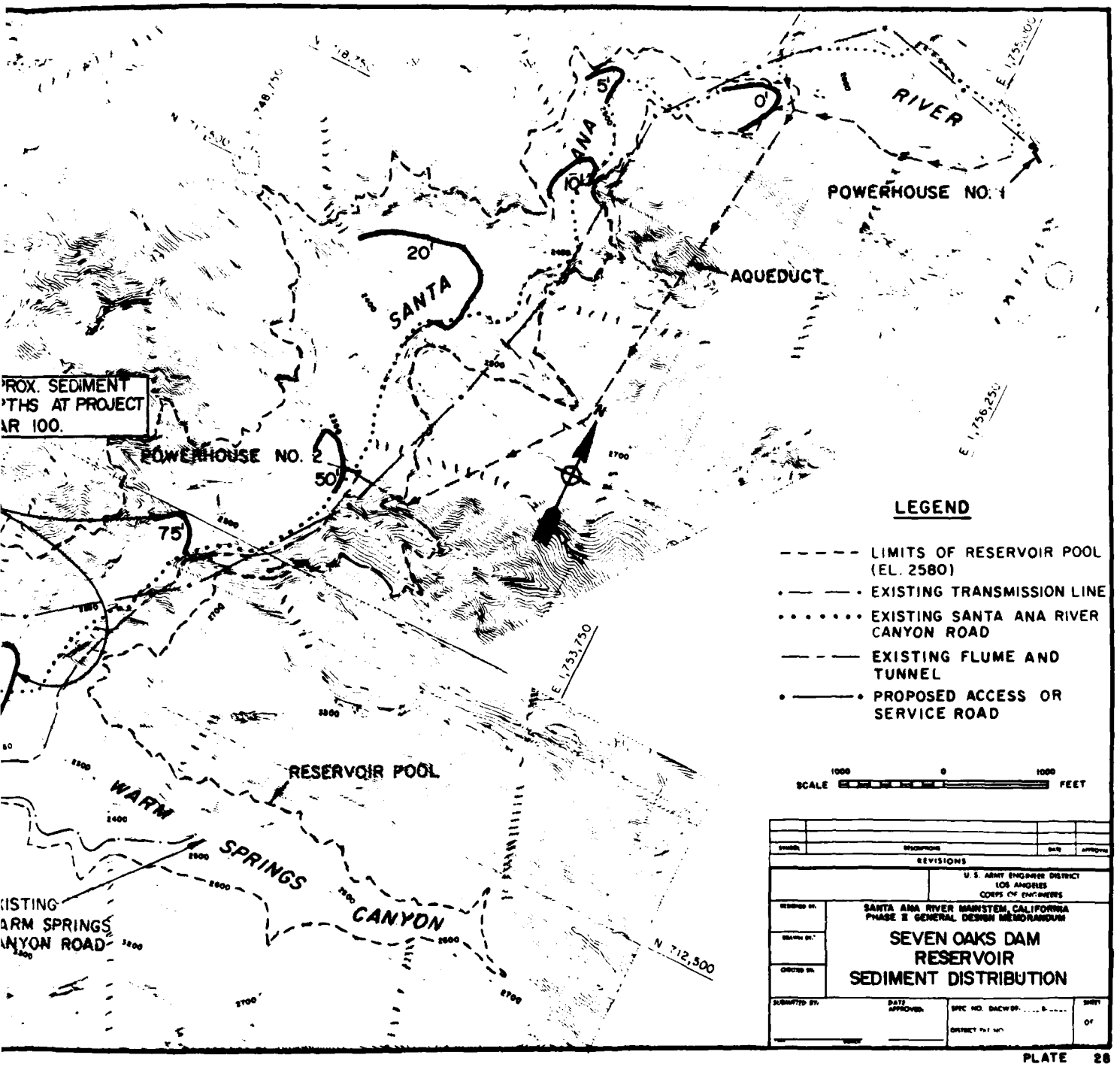
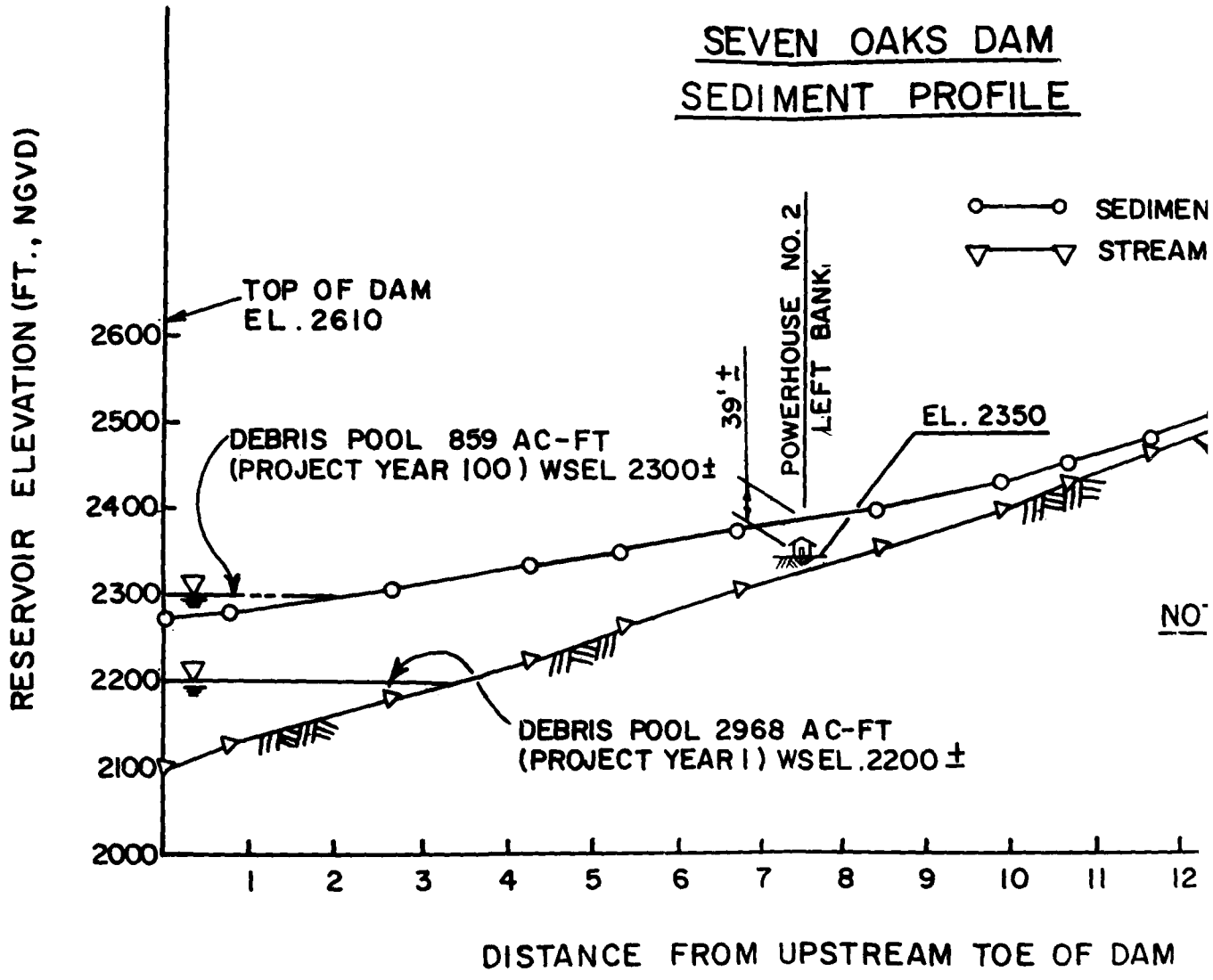
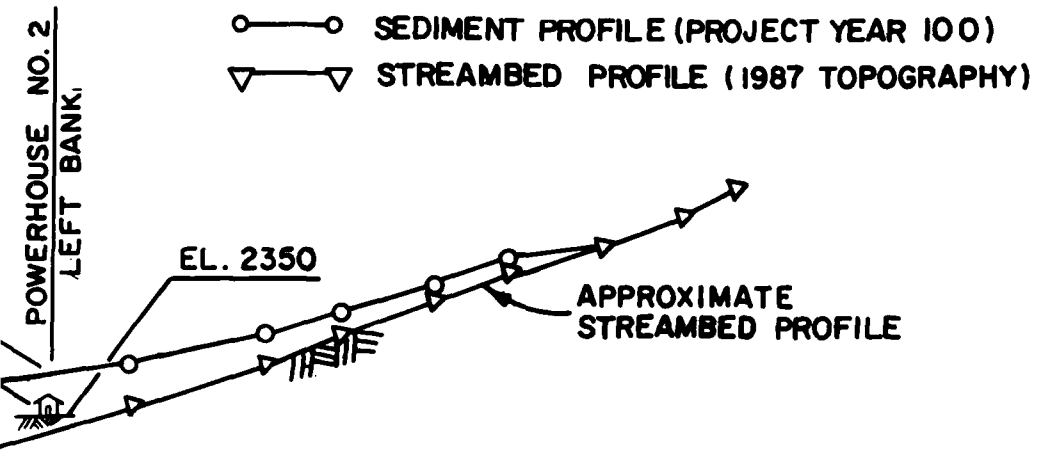

SANTA ANA RIVER
POWERHOUSE NO. 1
AQUEDUCT
POWERHOUSE NO. 2
RESERVOIR POOL
WARM SPRINGS CANYON
ROX. SEDIMENT
THS AT PROJECT
R 100.
ISTING
RM SPRINGS
NYON ROAD
LEGEND
LIMITS OF RESERVOIR POOL (EL. 2580)
EXISTING TRANSMISSION LINE
EXISTING SANTA ANA RIVER CANYON ROAD
EXISTING FLUME AND TUNNEL
PROPOSED ACCESS OR SERVICE ROAD
SCALE
FEET
REVISIONS
U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS
SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM
SEVEN OAKS DAM
RESERVOIR
SEDIMENT DISTRIBUTION

PLATE 28


SEVEN OAKS DAM
SEDIMENT PROFILE
SEDIMEN
STREAM
RESERVOIR ELEVATION (FT., NGVD)
TOP OF DAM
EL. 2610
POWERHOUSE NO. 2
LEFT BANK
39'±
EL. 2350
DEBRIS POOL 859 AC-FT
(PROJECT YEAR 100) WSEL 2300±
DEBRIS POOL 2968 AC-FT
(PROJECT YEAR 1) WSEL. 2200±
2600
2500
2400
2300
2200
2100
2000
1
2
3
4
5
6
7
8
9
10
11
12
DISTANCE FROM UPSTREAM TOE OF DAM

/EN OAKS DAM
IMENT PROFILE



NOTE: THE FLUMELINE TRESTLE ACROSS WARM SPRINGS CANYON WILL BE OVERTOPPED BY APPROXIMATELY 12 FEET OF SEDIMENT, UP TO EL. 2337±.

68 AC-FT
) WSEL.2200±

7 8 9 10 11 12 13 14 15 16 17

UPSTREAM TOE OF DAM (x1000 FT.)

DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

REVISIONS

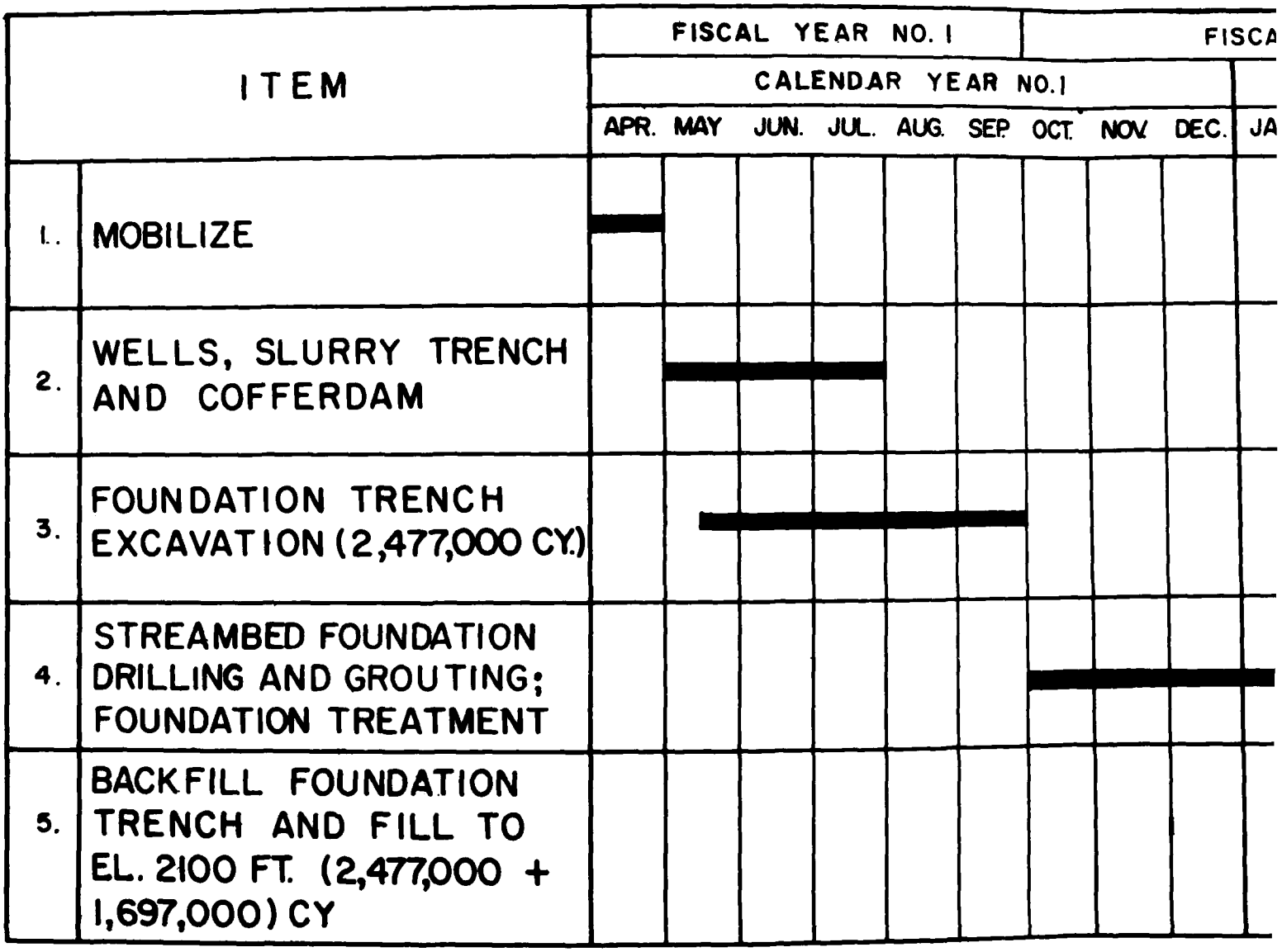
U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

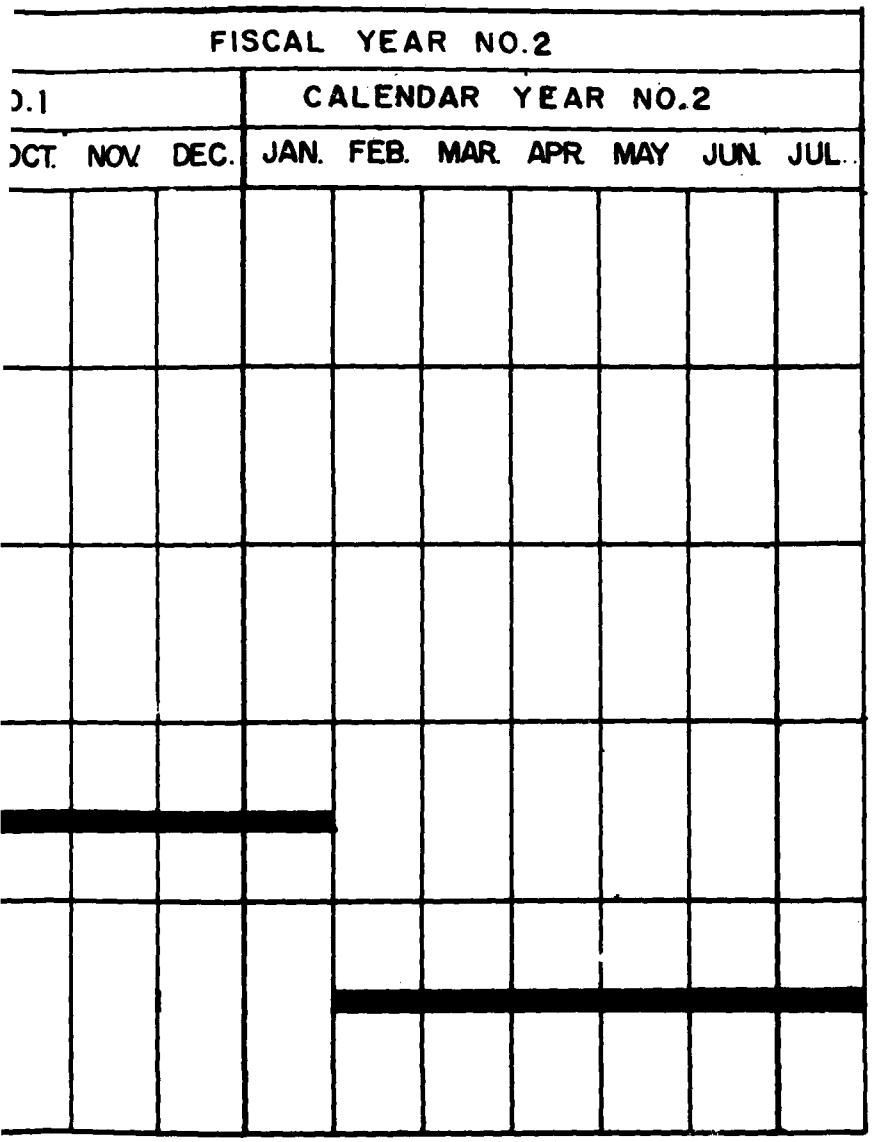
SEVEN OAKS DAM RESERVOIR SEDIMENT PROFILE

PLATE 59

MAIN EMBANKMENT CONSTRUCT



CONSTRUCTION YEARS



SYMBOL	DESCRIPTION	DATE	APPROVED

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY

DRAWN BY

CHECKED BY

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

EMBANKMENT

CONSTRUCTION SCHEDULE BELOW STREAMBED

SUBMITTED BY

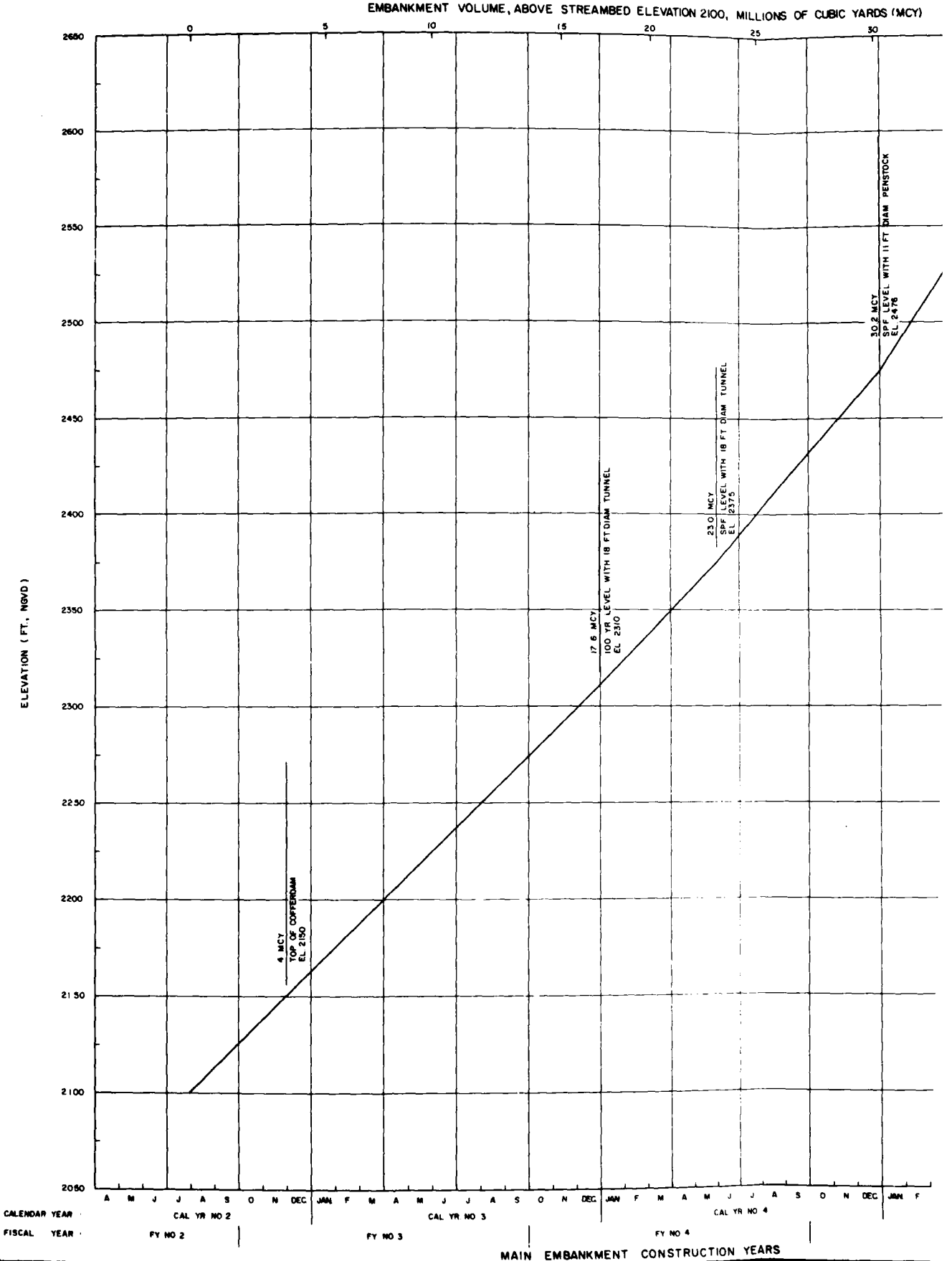
DATE APPROVED

SPEC. NO. DACW09- - -

DISTRICT FILE NO.

SHEET OF

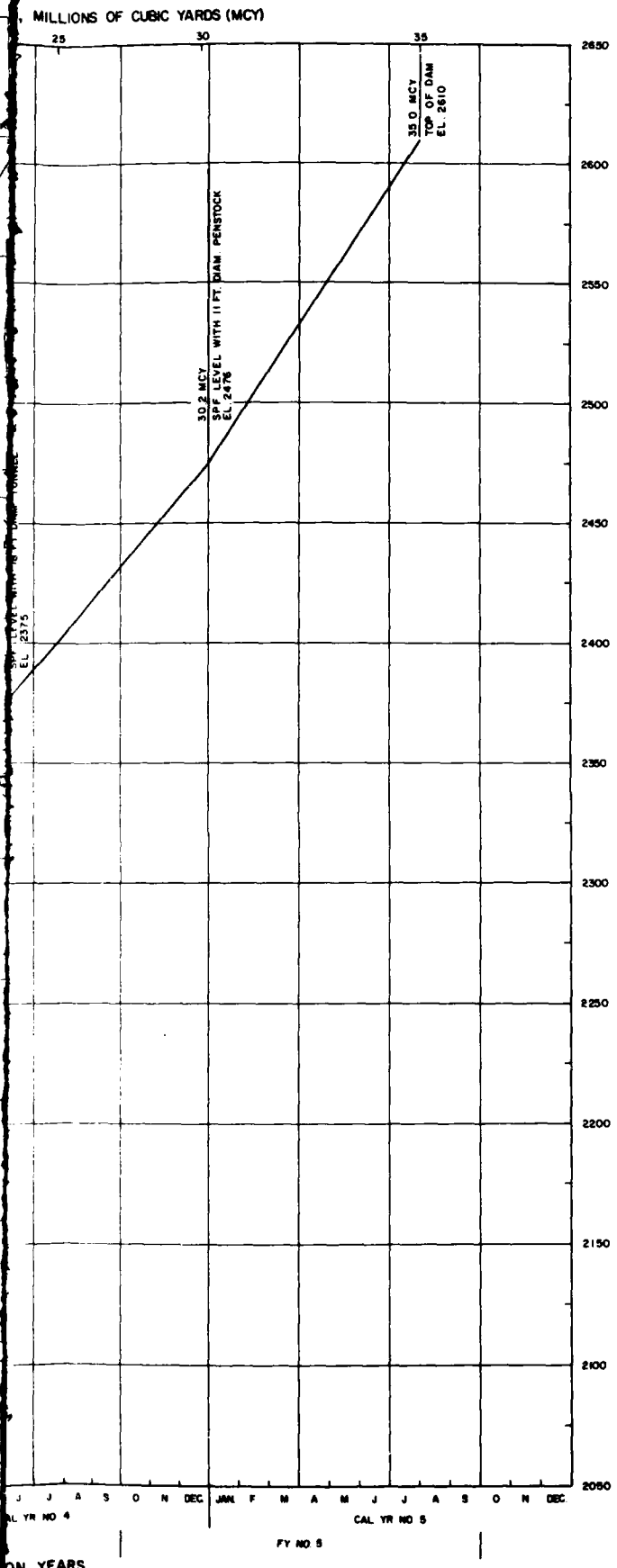
PLATE 30



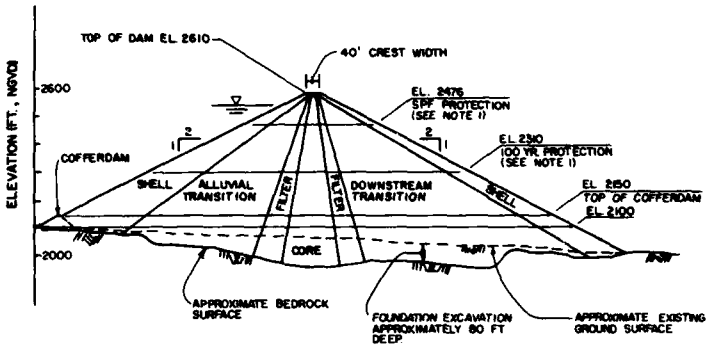
ENVIRONMENTAL
ENHANCEMENT
THRU ENGINEERING

PA

LUE ENGINEERING PAYS



SEVEN OAKS DAM
CROSS SECTION



EMBANKMENT CONSTRUCTION STAGES
NOT TO SCALE

NOTES:

1. THE WATER DIVERSION PLAN AND RELATED FLOOD PROTECTION LEVELS ARE BASED ON THE DOWNSTREAM CONTROL ALTERNATIVE FOR THE OUTLET WORKS TUNNEL. UPSTREAM CONTROL AND MIDTUNNEL CONTROL ALTERNATIVES WITH THEIR RESPECTIVE DIVERSION PLANS ARE PRESENTED IN THE FINAL PHASE II GDM. (ALSO SEE DISCUSSION IN SEVEN OAKS DAM, VOL. I, MAIN REPORT)
2. DIVERSION OF WATER WILL BE PROVIDED BY THE 18-FOOT DIAMETER OUTLET WORKS TUNNEL UNTIL JUNE OF FISCAL YEAR 4.
   DIVERSION FROM JUNE TO DECEMBER OF FISCAL YEAR 5 WILL UTILIZE THE OUTLET WORKS LOW FLOW CONDUITS.
   DIVERSION AFTER DECEMBER OF FISCAL YEAR 5 WILL UTILIZE THE OUTLET WORKS 11-FOOT DIAMETER PENSTOCK.
3. PRIMARY WINTER MONTHS ARE FROM NOVEMBER TO MARCH.
4. EMBANKMENT VOLUME BELOW ELEVATION 2100 IS APPROXIMATELY 4.5 MCY.
5. ELEVATION 2100 IS ASSUMED TO BE THE STREAMBED ELEVATION.

DATUM IS THE NATIONAL GEODETIC VERTICAL DATUM OF 1929

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY: DC
CHECKED BY:

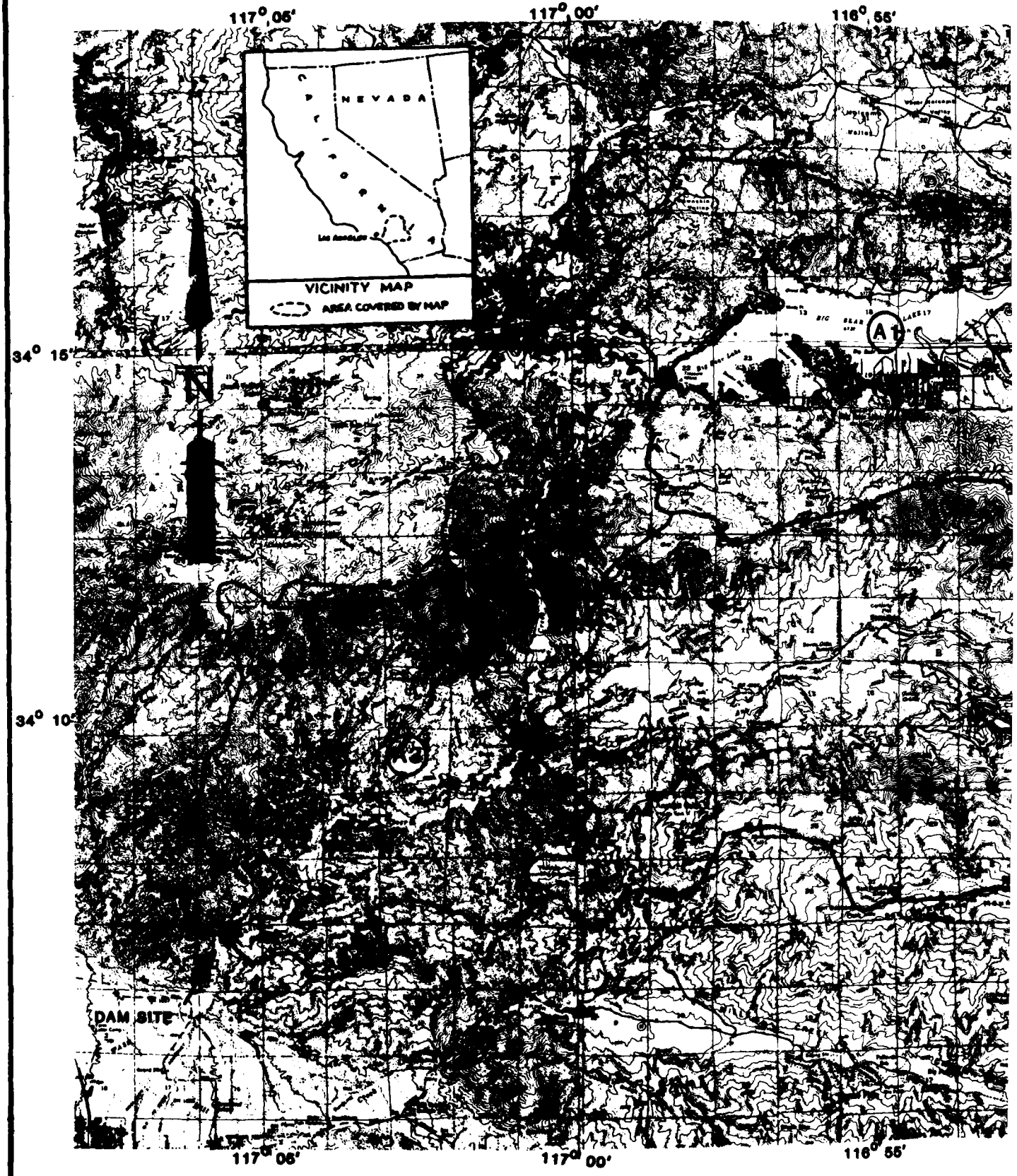
SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

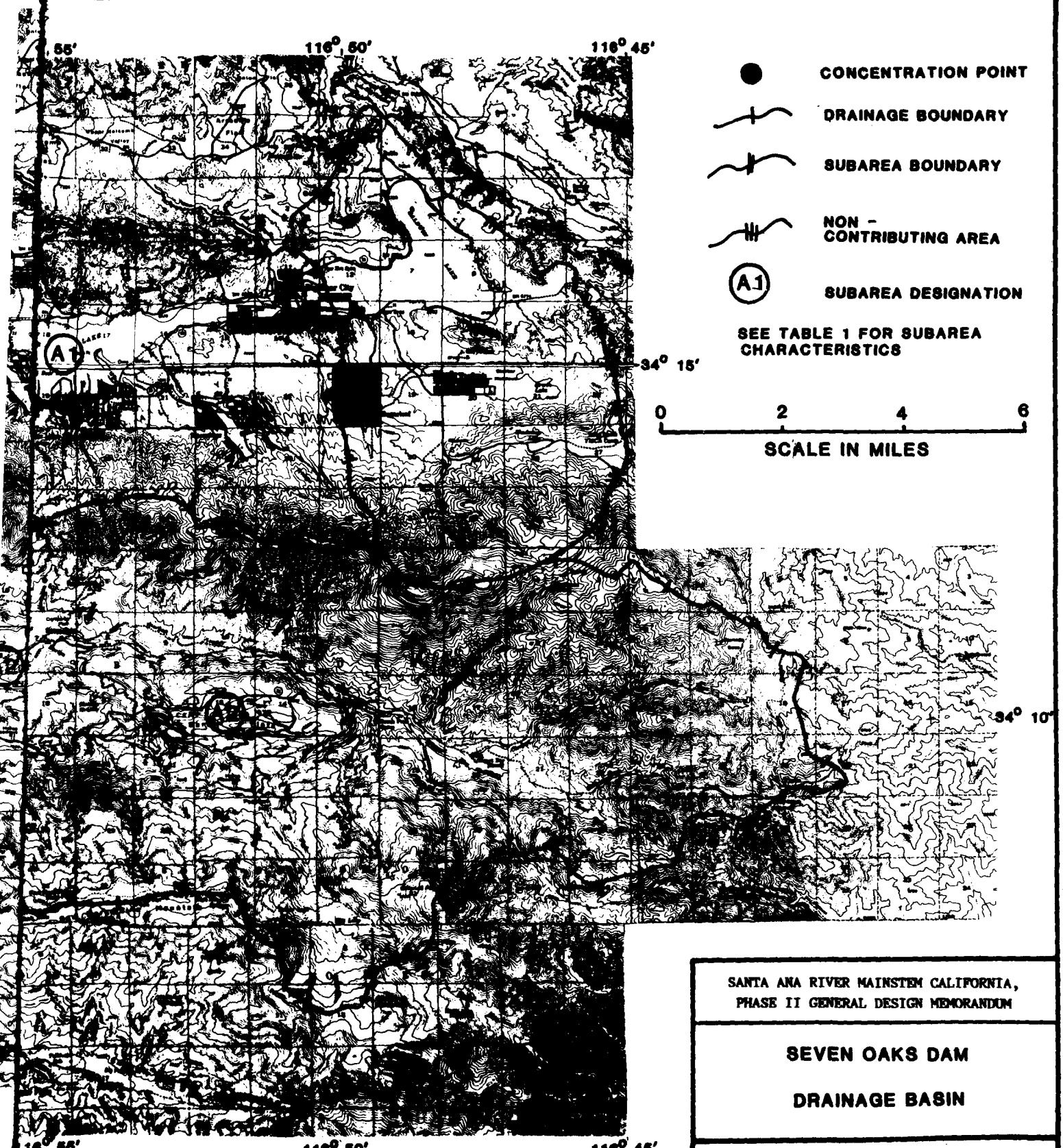
SEVEN OAKS DAM
EMBANKMENT
RATE OF PLACEMENT ABOVE STREAMBED

SUBMITTED BY: DATE APPROVED:
SPEC. NO. DACW09-____-____
DISTRICT FILE NO.
SHEET

PLATE 3

SAFETY PAYS


117° 06'
117° 00'
116° 55'
NEVADA
Los Angeles
VICINITY MAP
AREA COVERED BY MAP
34° 15'
34° 10'
DAM SITE
117° 06'
117° 00'
116° 55'



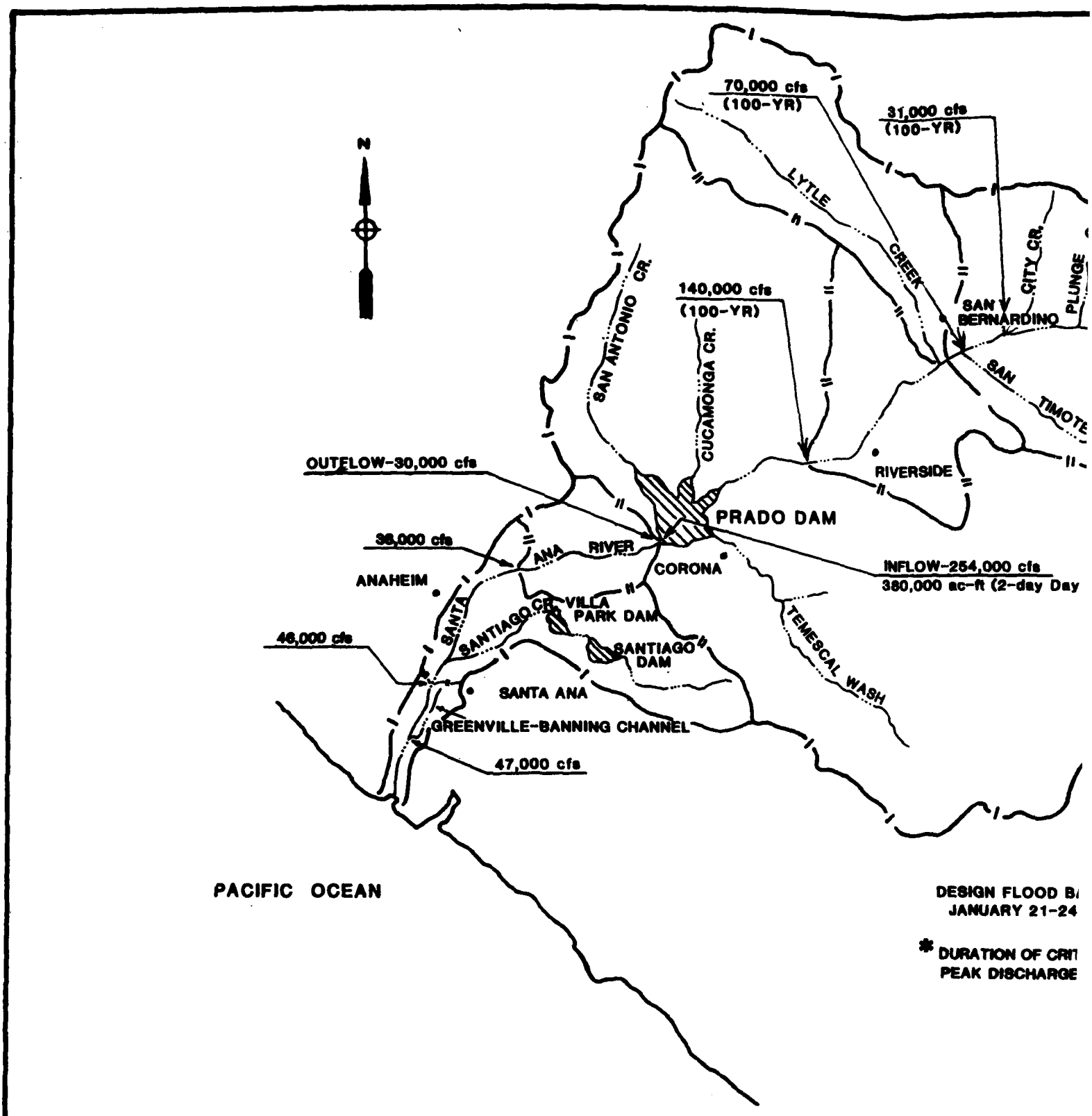
SANTA ANA RIVER MAINSTEM CALIFORNIA, PHASE II GENERAL DESIGN MEMORANDUM

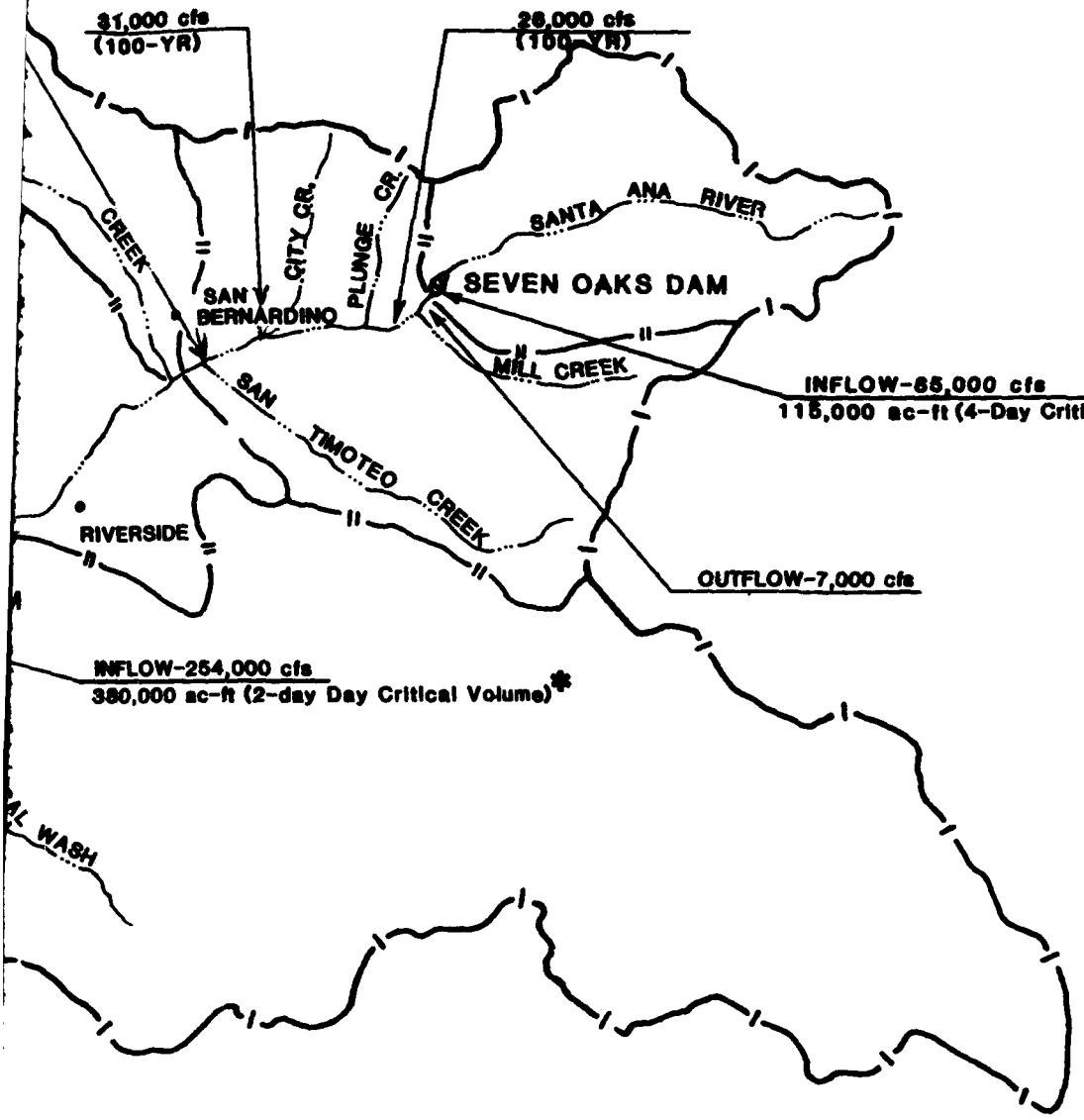
SEVEN OAKS DAM

DRAINAGE BASIN

US ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

PLATE 32


N
70,000 cfs
(100-YR)
31,000 cfs
(100-YR)
LYTLE
CREEK
CITY CR.
PLUNGE
SAN ANTONIO CR.
140,000 cfs
(100-YR)
CUCAMONGA CR.
SAN
BERNARDINO
SAN
TIMOTE
OUTFLOW-30,000 cfs
RIVERSIDE
PRADO DAM
36,000 cfs
ANA
RIVER
CORONA
INFLOW-254,000 cfs
380,000 ac-ft (2-day Day
ANAHEIM
SANTA
SANTIAGO CR.
VILLA
PARK DAM
46,000 cfs
SANTIAGO
DAM
TEMESCAL WASH
SANTA ANA
GREENVILLE-BANNING CHANNEL
47,000 cfs
PACIFIC OCEAN
DESIGN FLOOD B
JANUARY 21-24
* DURATION OF CRI
PEAK DISCHARGE



NOTES:

1. Reservoir Design Flood (RDF) at Seven Oaks Dam is based on NED storage.
2. Between Seven Oaks Dam and Prado, the Santa Ana River is subject to Flood Plain Management. Discharges are 100-year frequency.
3. RDF at Prado Dam is 92% of SPF. Based on 2+ day volume being the critical duration, this is a 190-year event.
4. Design flood from Prado Dam to the Pacific Ocean is a 190-year event.

DESIGN FLOOD BASED ON OCCURENCE OF JANUARY 21-24, 1943 GENERAL STORM.

* DURATION OF CRITICAL VOLUME IS THAT WHICH GENERATED PEAK DISCHARGE AND WATER SURFACE ELEVATION

SCALE IN MILES

5 0 5 10

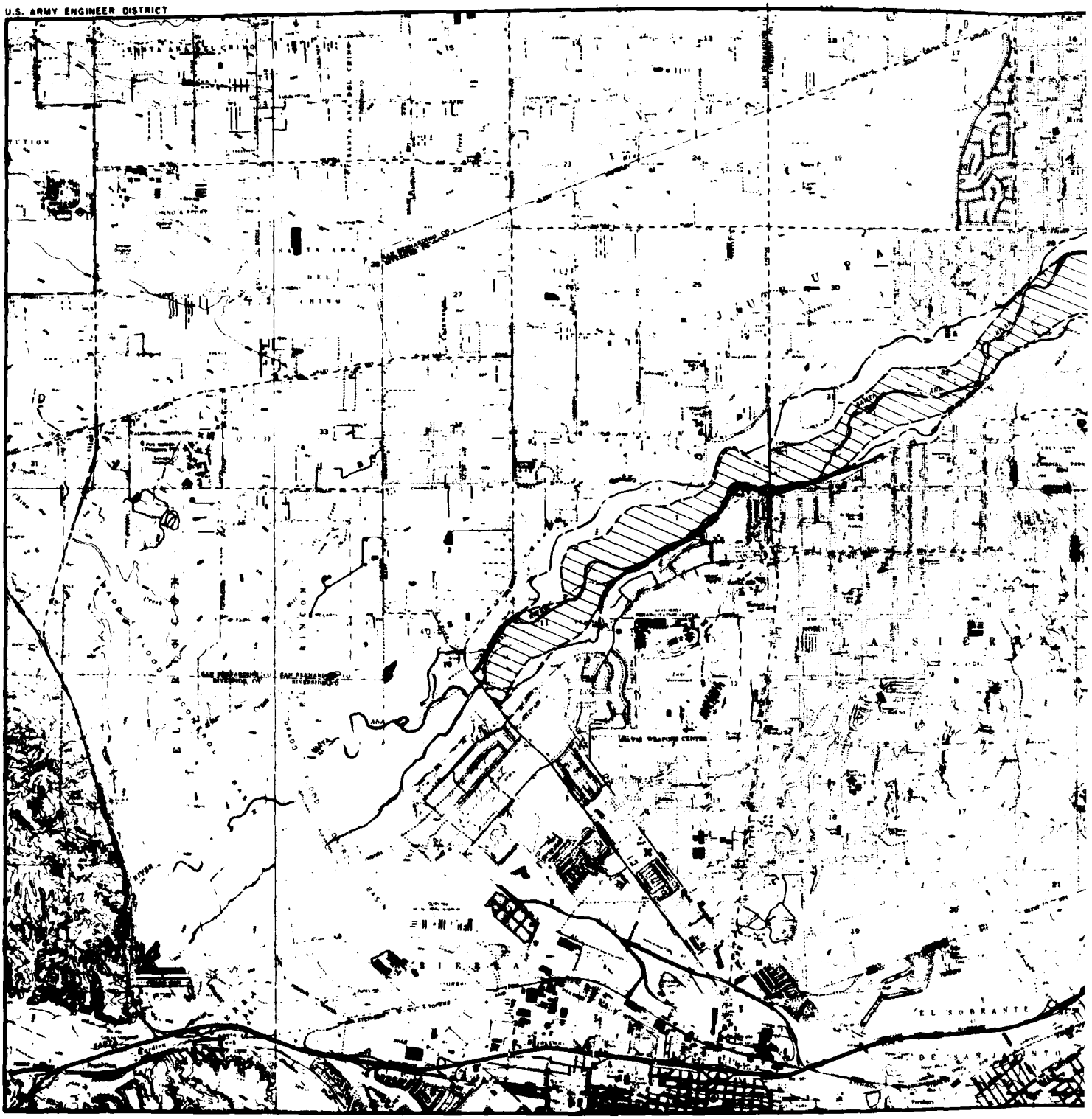
SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

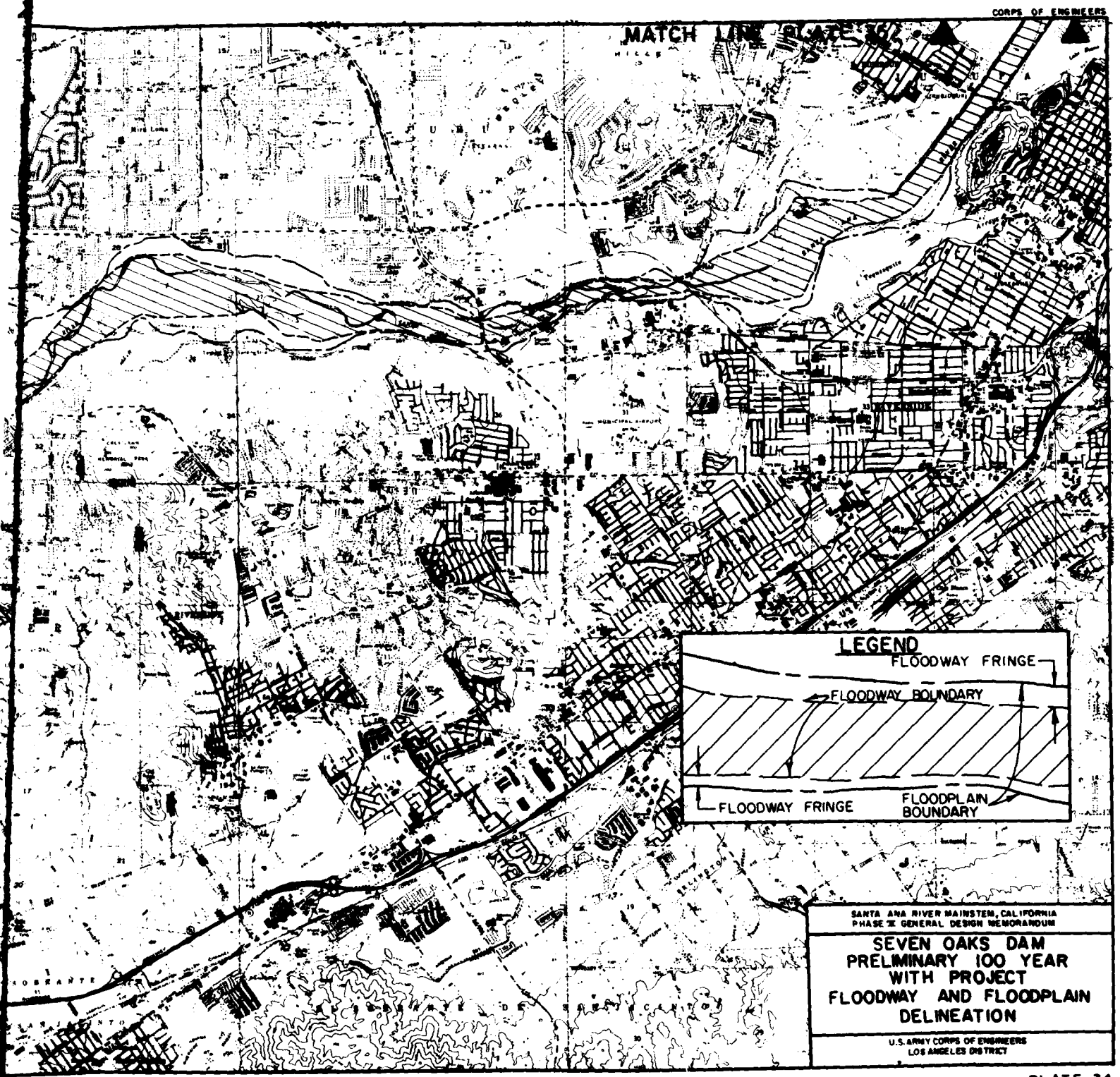
DESIGN FLOOD PEAK DISCHARGES
SANTA ANA RIVER
FUTURE CONDITIONS
WITH RECOMMENDED PLAN

US ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

PLATE 33

U.S. ARMY ENGINEER DISTRICT



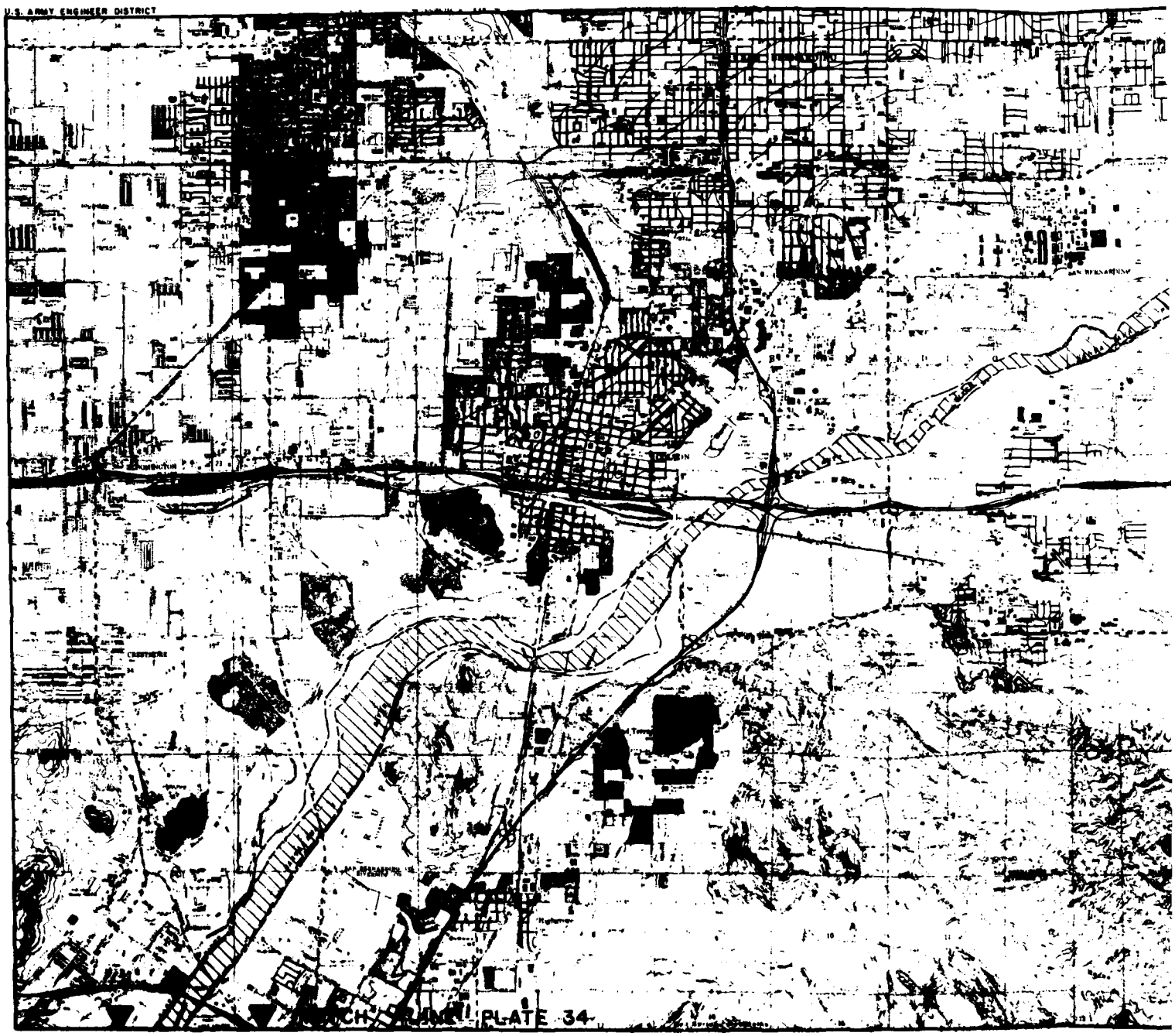


SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

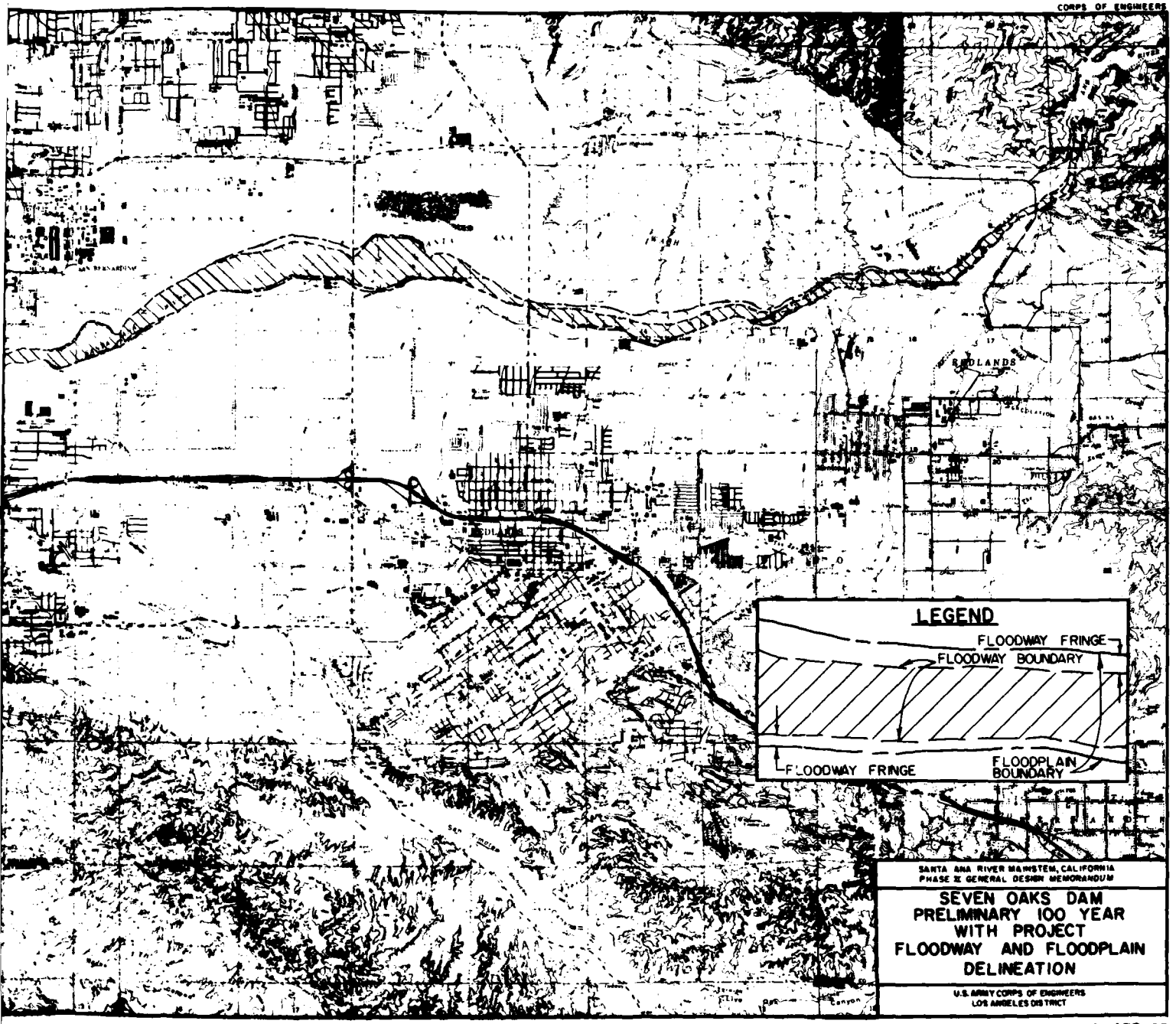
SEVEN OAKS DAM PRELIMINARY 100 YEAR WITH PROJECT FLOODWAY AND FLOODPLAIN DELINEATION

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

PLATE 34


U.S. ARMY ENGINEER DISTRICT
PLATE 34

CORPS OF ENGINEERS

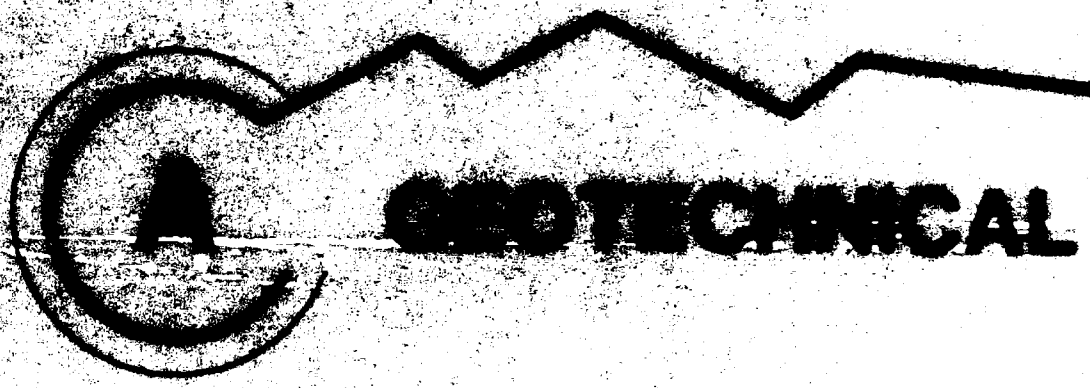


SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM PRELIMINARY 100 YEAR WITH PROJECT FLOODWAY AND FLOODPLAIN DELINEATION

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

PLATE 35


GEOTECHNICAL

Appendix A

GEOTECHNICAL

CONTENTS

		Page
I.	INTRODUCTION	A-I-1
	Purpose and Scope	A-I-1
	Organization of Appendix	A-I-1
	Project Features	A-I-2
	Location	A-I-2
II.	GEOLOGY	A-II-1
	Regional Geologic Setting	A-II-1
	Recent Geologic History and Tectonics	A-II-1
	San Andreas Fault System	A-II-2
	San Bernardino Mountains	A-II-3
	Local Geology and Topography	A-II-4
	Topography	A-II-4
	Rock Units	A-II-4
	Gneissic Rocks (gn)	A-II-4
	Quartz Diorite (qd)	A-II-5
	Quartz Monzonite (qm, qmp)	A-II-5
	Tertiary Sediments (Tmc)	A-II-5
	Alluvial/Colluvial Units	A-II-5
	Terrace Deposits (Qt)	A-II-5
	Colluvium (Qc)	A-II-6
	Landslide Debris	A-II-6
	Stream Terrace (Qst)	A-II-6
	Recent Alluvium (Qal)	A-II-6
	Faulting	A-II-7
III.	GROUNDWATER	A-III-1
	Surface Flow	A-III-1
	Subsurface Flow	A-III-1
	Groundwater Levels	A-III-2
IV.	SEISMICITY	A-IV-1
	Regional Seismicity	A-IV-1
	San Andreas Fault System	A-IV-1
	Other Major Faults	A-IV-2

Contents (Continued)

		Page
	Subsidiary Faulting	A-IV-2
	Earthquake History	A-IV-3
	Seismic Probability	A-IV-4
	Earthquake Ground Motion	A-IV-4
	Reservoir-Induced Seismicity	A-IV-5
	General	A-IV-5
	Seven Oaks Damsite	A-IV-6
V.	FIELD INVESTIGATIONS	A-V-1
	Scope of Investigations	A-V-1
	Investigation Procedures	A-V-2
	Geologic Mapping, Literature Research and Aerial Photo Interpretation	A-V-2
	Special Fault Studies - Soil Chronology	A-V-2
	Trenching	A-V-2
	Fault Trenching	A-V-3
	Foundation Bedrock	A-V-3
	Foundation Alluvium	A-V-3
	Pervious Borrow Area	A-V-3
	Impervious Borrow Area	A-V-4
	Drilling	A-V-4
	Rotary-Wash Drilling	A-V-4
	Observation Wells	A-V-4
	Geophysical Crossholes	A-V-5
	Core Drilling	A-V-5
	Core Logging	A-V-5
	Discontinuity Analysis	A-V-6
	Rock Quality Designation	A-V-6
	Weathering Index	A-V-6
	Point Load Testing	A-V-7
	Pressure Testing	A-V-7
	Bucket Auger Drilling	A-V-8
	Becker Hammer	A-V-8
	Geophysical Surveys	A-V-9
	Magnetometer	A-V-9
	Refractive Seismic	A-V-9
	Downhole Seismic	A-V-10
	Crosshole Seismic	A-V-10
	Summary of Explorations	A-V-10
	Streambed Foundation	A-V-10
	Drilling	A-V-11
	Geophysical Surveys	A-V-12
	Trenching	A-V-12
	Right Abutment	A-V-13
	Drilling	A-V-13
	Trenching	A-V-14
	Left Abutment	A-V-14
	Drilling	A-V-14
	Geophysical Surveys	A-V-14

Contents (Continued)

		Page
	Trenching	A-V-15
	Spillway	A-V-15
	Drilling	A-V-15
	Geophysical Surveys	A-V-16
	Trenching	A-V-16
	Government Canyon Ridge	A-V-16
	Drilling	A-V-16
	Geophysical Surveys	A-V-17
	Trenching	A-V-17
	Outlet Works	A-V-17
	Drilling	A-V-17
	Geophysical Surveys	A-V-18
	Trenching	A-V-18
	Downstream Pervious Borrow Area	A-V-18
	Impervious Borrow Area	A-V-19
	Fault Investigations	A-V-19
	Feature Design Investigations	A-V-21
	Streambed Foundation	A-V-21
	Right Abutment	A-V-21
	Left Abutment	A-V-21
	Spillway	A-V-22
	Government Canyon Ridge	A-V-22
	Outlet Works	A-V-22
	Downstream Pervious Borrow Area	A-V-22
	Impervious Borrow Area	A-V-23
VI.	LABORATORY TESTING PROGRAM FOR ROCK QUALITY	A-VI-1
	Purpose and Scope	A-VI-1
	Unconfined Compressive Strength	A-VI-1
	Rock Quality Tests	A-VI-2
	Sampling Procedure	A-VI-2
VII.	RESULTS OF INVESTIGATIONS AND TESTING	A-VII-1
	Core Drilling - General	A-VII-1
	Streambed Foundation Bedrock	A-VII-1
	Drilling Summary	A-VII-1
	Geophysical Survey Results	A-VII-2
	Refractive Seismic	A-VII-3
	Downhole Seismic	A-VII-3
	Laboratory Test Results	A-VII-3
	Unconfined Compressive Strength	A-VII-3
	Rock Quality	A-VII-4
	Petrographic Analysis	A-VII-4
	Foundation Alluvium	A-VII-5
	Field Tests and Results	A-VII-5
	Refractive Seismic Surveys	A-VII-5
	Crosshole and Downhole Geophysical Surveys	A-VII-6
	Large-Scale Density Tests	A-VII-6
	Aquifer Test	A-VII-7

Contents (Continued)

	Page
Laboratory Tests and Results	A-VII-7
Mechanical Analysis	A-VII-7
Maximum Density	A-VII-8
Triaxial Compression	A-VII-11
Government Canyon Ridge	A-VII-11
Mapping	A-VII-11
Drilling Summary	A-VII-11
Geophysical Survey Results	A-VII-12
Refractive Seismic	A-VII-12
Downhole Seismic	A-VII-12
Laboratory Test Results	A-VII-13
Unconfined Compressive Strength	A-VII-13
Rock Quality	A-VII-13
Petrographic Analysis	A-VII-14
Spillway	A-VII-14
Mapping	A-VII-14
Drilling Summary	A-VII-15
Geophysical Survey Results	A-VII-16
Refractive Seismic	A-VII-16
Downhole Seismic	A-VII-16
Laboratory Test Results	A-VII-17
Unconfined Compressive Strength	A-VII-17
Rock Quality	A-VII-17
Petrographic Analysis	A-VII-18
Right Abutment	A-VII-19
Mapping	A-VII-19
Drilling Summary	A-VII-20
Geophysical Survey Results	A-VII-20
Laboratory Test Results	A-VII-20
Unconfined Compressive Strength	A-VII-20
Rock Quality	A-VII-21
Petrographic Analysis	A-VII-21
Left Abutment	A-VII-21
Mapping	A-VII-21
Drilling Summary	A-VII-22
Core Drilling	A-VII-22
Bucket Auger Drilling	A-VII-22
Geophysical Survey Results	A-VII-23
Refractive Seismic	A-VII-23
Downhole Seismic	A-VII-23
Laboratory Rock Test Results	A-VII-24
Unconfined Compressive Strength	A-VII-24
Rock Quality	A-VII-24
Petrographic Analysis	A-VII-24
Laboratory Soil Tests and Results	A-VII-25
Soil Classification Tests	A-VII-25
Moisture-Density Relationships	A-VII-25

Contents (Continued)

	Page
Outlet Works	A-VII-25
Mapping	A-VII-25
Drilling Summary	A-VII-26
Core and Rotary-Wash Drilling	A-VII-26
Bucket Auger Drilling	A-VII-27
Geophysical Survey Results	A-VII-27
Refractive Seismic	A-VII-27
Inlet Portal	A-VII-28
Outlet Portal	A-VII-28
Energy Dissipator	A-VII-28
Downhole Seismic	A-VII-29
Laboratory Test Results	A-VII-29
Unconfined Compressive Strength	A-VII-29
Rock Quality	A-VII-29
Petrographic Analysis	A-VII-30
Reservoir Area	A-VII-30
Mapping	A-VII-30
Downstream Pervious Borrow Area	A-VII-30
Field Tests and Results	A-VII-30
Mass Gradation	A-VII-30
Oversize Relations	A-VII-31
Laboratory Tests and Results	A-VII-31
Mechanical Analysis	A-VII-31
Specific Gravity	A-VII-32
Maximum Density	A-VII-32
Triaxial Compression	A-VII-33
Permeability	A-VII-33
Upstream Pervious Borrow Area	A-VII-33
Field Tests and Results	A-VII-33
Laboratory Tests and Results	A-VII-34
Mechanical Analysis	A-VII-34
Specific Gravity	A-VII-34
Impervious Borrow Area	A-VII-34
Field Tests and Results	A-VII-34
Sand Cone Density Tests	A-VII-34
Laboratory Tests and Results	A-VII-35
Mechanical Analysis	A-VII-35
Atterberg Limits	A-VII-35
Bentonite Blending	A-VII-35
Moisture-Density Relationships	A-VII-36
Consolidation	A-VII-37
Permeability	A-VII-37
Triaxial Compression	A-VII-37
Dispersion	A-VII-37
Specific Gravity	A-VII-37
North Branch Fault Investigations	A-VII-38

Contents (Continued)

		Page
VIII.	ANALYSIS OF DATA	A-VIII-1
	Streambed Foundation Bedrock	A-VIII-1
	Depth and Configuration of Bedrock Surface	A-VIII-1
	Bedrock Characteristics	A-VIII-1
	Faulting	A-VIII-2
	Streambed Foundation Alluvium	A-VIII-2
	Gradation	A-VIII-3
	Density	A-VIII-3
	Liquefaction Potential	A-VIII-4
	Right Abutment	A-VIII-5
	Overburden Properties	A-VIII-5
	Bedrock Properties	A-VIII-5
	Faulting	A-VIII-6
	Left Abutment	A-VIII-6
	Overburden Properties	A-VIII-6
	Bedrock Properties	A-VIII-6
	Faulting	A-VIII-7
	Spillway	A-VIII-7
	Overburden Properties	A-VIII-7
	Bedrock Properties	A-VIII-8
	Excavatability	A-VIII-8
	Erodibility	A-VIII-8
	Rock Quality/Size	A-VIII-8
	Cut-Slope Stability	A-VIII-9
	Faulting	A-VIII-9
	Government Canyon Drainage Channel	A-VIII-10
	Overburden Properties	A-VIII-10
	Bedrock Properties	A-VIII-10
	Excavatability	A-VIII-10
	Rock Quality/Size	A-VIII-10
	Cut-Slope Stability	A-VIII-11
	Spillway/Government Canyon Ridge Rock Quality Comparison	A-VIII-11
	Outlet Works	A-VIII-12
	Overburden Properties	A-VIII-12
	Bedrock Configuration	A-VIII-12
	Bedrock Properties	A-VIII-13
	Excavatability	A-VIII-13
	Tunnel	A-VIII-13
	Portals	A-VIII-13
	Channels	A-VIII-13
	Tunnel Rockmass Classification	A-VIII-14
	Slope Stability	A-VIII-14
	Faulting	A-VIII-14
	Reservoir Area-Landslide Potential	A-VIII-15
	Downstream Pervious Borrow Area	A-VIII-16
	Alluvial Transition Material	A-VIII-17
	Filter Material	A-VIII-17
	Shell Material	A-VIII-17

Contents (Continued)

		Page
	Upstream Pervious Borrow Area	A-VIII-18
	Impervious Borrow Area	A-VIII-19
	Material Distribution	A-VIII-19
	Stress-Strain Characteristics	A-VIII-19
IX.	DESIGN VALUES	A-IX-1
	General	A-IX-1
	Foundation	A-IX-1
	Bedrock	A-IX-1
	Alluvium	A-IX-2
	Embankment	A-IX-3
	Impervious Core	A-IX-3
	Filter	A-IX-4
	Alluvial Transition and Shell	A-IX-4
	Downstream Transition	A-IX-4
	Feature Design Testing	A-IX-5
X.	DESIGN AND ANALYSIS	A-X-1
	Embankment	A-X-1
	Seismic Criteria	A-X-1
	Maximum Credible Event	A-X-1
	Site Ground Motions	A-X-1
	Design Earthquake Accelerogram	A-X-2
	Defensive Design Measures	A-X-3
	Embankment Section	A-X-3
	Zonation	A-X-3
	Cofferdam	A-X-4
	Slurry Trench Cutoff	A-X-4
	Slope Stability	A-X-5
	Static	A-X-5
	End of Construction	A-X-6
	Sudden Drawdown	A-X-6
	Partial Pool	A-X-6
	Steady Seepage	A-X-6
	Earthquake	A-X-7
	Finite Element Method Analysis	A-X-7
	Dynamic	A-X-8
	Finite Element Method Analysis	A-X-8
	Permanent Deformation Analysis	A-X-8
	Settlement	A-X-10
	Total Settlement	A-X-10
	Differential Settlement	A-X-11
	Foundation	A-X-11
	Seepage	A-X-11
	Embankment	A-X-12
	Case I: Steady-State Seepage with the Reservoir Pool at Spillway Crest Elevation (Intact Core)	A-X-13

Contents (Continued)

		Page
	Case II: Steady-State Seepage with the Reservoir Pool at Spillway Crest Elevation (Cracked Core)..	A-X-13
	Case III: Seepage with the Reservoir at Debris Pool Elevation.....	A-X-13
	Foundation.....	A-X-14
	Slope Protection.....	A-X-14
	Spillway.....	A-X-15
	Cut-Slope Configuration.....	A-X-15
	Invert Sill.....	A-X-15
	Drainage Channel.....	A-X-16
	Outlet Works.....	A-X-16
	Portals.....	A-X-16
	Cut-Slope Configuration.....	A-X-16
	Slope Reinforcement.....	A-X-16
	Drains.....	A-X-17
	Tunnel.....	A-X-17
	Configuration.....	A-X-17
	Support.....	A-X-17
	Temporary.....	A-X-17
	Permanent.....	A-X-18
	Rupture.....	A-X-18
	Consolidated Grouting.....	A-X-18
	Channels and Energy Dissipator.....	A-X-18
	Configuration.....	A-X-18
	Slope Protection.....	A-X-19
	Summary.....	A-X-19
XI.	CONSTRUCTION MATERIALS.....	A-XI-1
	Embankment Materials.....	A-XI-1
	Core.....	A-XI-1
	Filter.....	A-XI-1
	Alluvial Transition.....	A-XI-1
	Downstream Transition.....	A-XI-2
	Shell.....	A-XI-2
	Concrete Materials.....	A-XI-2
	General.....	A-XI-2
	Climatic Conditions.....	A-XI-3
	Sources of Cement.....	A-XI-4
	Sources of Pozzolans.....	A-XI-4
	Aggregates.....	A-XI-5
	Off-Site Sources.....	A-XI-5
	On-Site Source.....	A-XI-5
	Geologic Aspects of Aggregate Sources.....	A-XI-5
	Geologic Aspects of On-Site Source.....	A-XI-6
	Geologic Aspects of Off-Site Sources.....	A-XI-6
	Lytle Creek.....	A-XI-6
	Santa Ana River.....	A-XI-6
	San Gorgonio River.....	A-XI-7

		Page
	Trenching	A-XI-7
	First Stage Crushing	A-XI-7
	Summary of Aggregate Test Data	A-XI-8
	On-Site Source	A-XI-8
	Owl Rock Products	A-XI-10
	4th Street Crusher	A-XI-10
	Beaumont Concrete Company	A-XI-14
	C.L. Pharris	A-XI-16
	Water	A-XI-17
	Admixtures	A-XI-18
	Recommendations	A-XI-18
	Laboratory Studies	A-XI-18
	Mix Designs	A-XI-19
	Specification Requirements	A-XI-19
XII.	CONSTRUCTION PROCEDURES AND REQUIREMENTS	A-XII-1
	Diversion and Control of Water	A-XII-1
	Slurry Trench Cutoff	A-XII-1
	Cofferdam	A-XII-2
	Dewatering Wells	A-XII-2
	Outlet Works	A-XII-2
	Portals	A-XII-2
	Excavation	A-XII-2
	Reinforcement	A-XII-3
	Channels and Energy Dissipator	A-XII-3
	Excavation	A-XII-3
	Disposition of Materials	A-XII-3
	Tunnel and Shaft	A-XII-4
	Excavation	A-XII-4
	Support	A-XII-4
	Grouting	A-XII-4
	Disposition of Materials	A-XII-5
	Foundation Preparation	A-XII-5
	Abutment Excavation	A-XII-5
	Right Abutment	A-XII-5
	Left Abutment	A-XII-6
	Streambed Foundation Excavation	A-XII-6
	Bedrock Surface Cleaning	A-XII-7
	Curtain Grouting	A-XII-7
	Bedrock Surface Treatment	A-XII-7
	Spillway	A-XII-8
	General Excavation	A-XII-8
	Slope and Invert Controlled Excavation	A-XII-8
	Disposition of Materials	A-XII-8
	Slope Reinforcement	A-XII-8
	Sill Construction	A-XII-8
	Drainage Channel	A-XII-9

Contents (Continued)

		Page
	Embankment	A-XII-9
	Impervious Core	A-XII-9
	Material Properties	A-XII-9
	Excavation, Transportation, and Processing	A-XII-10
	Lift Thickness and Compaction	A-XII-10
	Filter	A-XII-10
	Material Properties	A-XII-10
	Excavation, Transportation, and Processing	A-XII-10
	Lift Thickness and Compaction	A-XII-12
	Alluvial Transition	A-XII-12
	Material Properties	A-XII-12
	Excavation, Transportation, and Processing	A-XII-12
	Lift Thickness and Compaction	A-XII-12
	Downstream Transition	A-XII-13
	Material Properties	A-XII-13
	Excavation, Transportation, and Processing	A-XII-13
	Lift Thickness and Compaction	A-XII-13
	Shell	A-XII-13
	Material Properties	A-XII-14
	Excavation, Transportation, and Processing	A-XII-14
	Lift Thickness and Compaction	A-XII-14
	Roads	A-XII-14
	Construction Roads	A-XII-14
	Truck Transportation and Haul Roads	A-XII-14
	Belt Transportation Maintenance Roads	A-XII-15
	Permanent Access Roads	A-XII-15
	Materials	A-XII-15
	Placement Requirements	A-XII-15
	Relocations	A-XII-15
	Roads	A-XII-15
	Powerlines	A-XII-15
XIII.	CONSTRUCTION SCHEDULE	A-XIII-1
	Test Excavation and Test Fills	A-XIII-1
	General	A-XIII-1
	Dam Foundation Investigations	A-XIII-2
	Alluvial Test Fill	A-XIII-3
	Test Excavation and Rock Test Fill	A-XIII-3
	Outlet Works Portal Excavation	A-XIII-5
	Outlet Works Adits	A-XIII-5
	Streambottom Equipment Access Road	A-XIII-5
	Outlet Works	A-XIII-6
	Abutment Excavation	A-XIII-6
	Embankment and Spillway	A-XIII-6
	Diversion and Control of Water	A-XIII-6
	Foundation Excavation	A-XIII-7
	Foundation Treatment	A-XIII-7
	Below Streambed Embankment Fill	A-XIII-7
	Above Streambed Embankment Fill	A-XIII-7

Contents (Continued)

		Page
XIV.	QUALITY CONTROL	A-XIV-1
	Quality Control Testing Program	A-XIV-1
	Construction Quality Control Testing	A-XIV-1
	Contractor Quality Control	A-XIV-1
	Government Quality Assurance	A-XIV-1
	Government Staff	A-XIV-2
	Contract Testing Laboratory	A-XIV-2
	Tests	A-XIV-2
	Core	A-XIV-3
	Filter	A-XIV-3
	Alluvial Transition	A-XIV-3
	Downstream Transition	A-XIV-3
	Shell	A-XIV-4
	Reporting	A-XIV-4
	Monitoring	A-XIV-4
	Record Testing	A-XIV-4
XV.	INSTRUMENTATION	A-XV-1
	Preconstruction Microseismic Network	A-XV-1
	Embankment	A-XV-1
	Settlement	A-XV-1
	Horizontal Movement	A-XV-2
	Pore Pressures	A-XV-2
	Seepage	A-XV-2
	Seismic Motions	A-XV-2
	Monitoring Program	A-XV-3
	Evaluation During Construction	A-XV-3
	Verification of Design Assumptions	A-XV-3
	Post-Construction F.E.M. Evaluation	A-XV-3
	Post-Construction Performance Monitoring	A-XV-4
XVI.	REFERENCES	A-XVI-1

Tables

No.	Title	
A-1	Groundwater Levels in Observation Wells	A-III-2
A-2	Estimated (Mean) Ground Motion Parameters on Rock at Site	A-IV-5
A-3	Classification of Degree of Weathering of Rocks	A-V-6
A-4	Streambed Drilling	A-V-11
A-5	Right Abutment Drilling	A-V-13
A-6	Left Abutment Drilling	A-V-14
A-7	Spillway Drilling	A-V-15
A-8	Government Canyon Ridge Drilling	A-V-16
A-9	Outlet Works - Inlet Portal Drilling	A-V-17
A-10	Outlet Works - Outlet Portal Drilling	A-V-18

Page

Tables (Continued)

No.	Title	
A-11	Outlet Works - Energy Dissipator Drilling.....	A-V-18
A-12	Streambed Drilling Summary.....	A-VII-2
A-13	Streambed, Results of Refractive Seismic Surveys.....	A-VII-3
A-14	Streambed Foundation Bedrock, Results of Compressive Strength Tests.....	A-VII-4
A-15	Streambed Foundation Bedrock, Results of Rock Quality Tests.....	A-VII-4
A-16	Results of Refractive Seismic Surveys in Foundation Alluvium (lines 84-1 through 84-5).....	A-VII-5
A-17	Laboratory Maximum Densities of Foundation Alluvium....	A-VII-10
A-18	Government Canyon Ridge, Drilling Summary.....	A-VII-11
A-19	Government Canyon Ridge, Results of Refractive Seismic Survey.....	A-VII-12
A-20	Government Canyon Ridge, Results of Downhole Geophysical Surveys.....	A-VII-12
A-21	Government Canyon Ridge, Results of Compressive Strength Tests.....	A-VII-13
A-22	Government Canyon Ridge, Results of Rock Quality Tests.	A-VII-13
A-23	Government Canyon Ridge, Results of Petrographic Analysis.....	A-VII-14
A-24	Spillway Drilling Summary.....	A-VII-15
A-25	Spillway, Results of Refractive Seismic Surveys.....	A-VII-16
A-26	Spillway, Results of Downhole Geophysical Surveys.....	A-VII-16
A-27	Spillway, Results of Compressive Strength Tests.....	A-VII-17
A-28	Spillway, Results of Rock Quality Tests.....	A-VII-18
A-29	Spillway, Results of Petrographic Analyses.....	A-VII-19
A-30	Right Abutment, Drilling Summary.....	A-VII-20
A-31	Right Abutment, Results of Compressive Strength Tests..	A-VII-21
A-32	Left Abutment, Core Drilling Summary.....	A-VII-22
A-33	Left Abutment, Bucket Auger Drilling Summary.....	A-VII-23
A-34	Left Abutment, Results of Refractive Seismic Surveys...	A-VII-23
A-35	Left Abutment, Results of Compressive Strength Tests...	A-VII-24
A-36	Left Abutment, Results of Petrographic Analyses.....	A-VII-25
A-37	Inlet Portal, Drilling Summary.....	A-VII-26
A-38	Outlet Portal, Drilling Summary.....	A-VII-27
A-39	Energy Dissipator, Drilling Summary.....	A-VII-27
A-40	Outlet Works - Inlet Portal, Results of Refractive Seismic Surveys.....	A-VII-28
A-41	Outlet Works - Outlet Portal, Results of Refractive Seismic Surveys.....	A-VII-28

Page

Tables (Continued)

No.	Title	
A-42	Outlet Works - Engery Dissipator, Results of Refractive Seismic Survey	A-VII-29
A-43	Outlet Works - Inlet Portal, Results of Compressive Strength Tests	A-VII-29
A-44	Outlet Works - Inlet Portal, Results of Rock Quality Tests	A-VII-30
A-45	Laboratory Maximum Densities of Downstream Pervious Borrow Area Materials	A-VII-32
A-46	Overall Drilling Summary, Streambed Foundation Bedrock.	A-VIII-2
A-47	Streambed Foundation/Left Abutment, Drilling Summary Comparison	A-VIII-7
A-48	Estimated Spillway Rock Gradation	A-VIII-9
A-49	Estimated Government Canyon Ridge Rock Gradation	A-VIII-11
A-50	Spillway/Government Canyon Ridge, Rockmass Comparison..	A-VIII-12
A-51	Downstream Pervious Borrow Area, Soil Investigation Summary	A-VIII-17
A-52	Upstream Pervious Borrow Area, Soil Investigation Summary	A-VIII-19
A-53	Impervious Core Design Strengths	A-IX-3
A-54	Adopted Design Values	A-IX-5
A-55	Scheduled FDM Laboratory Testing	A-IX-6
A-56	Computed Safety Factors for Static Loading	A-X-5
A-57	Computed Safety Factors for Seismic Loading	A-X-7
A-58	Computed Safety Factor for Seismic Loading Based on WES Procedure (WES Misc. Paper GL-84-13)	A-X-10
A-59	Effective Hydraulic Conductivity Values	A-X-13
A-60	Minimum Revetment Design	A-X-15
A-61	Estimated Concrete Quantities	A-XI-3
A-62	Physical Tests on Concrete Aggregates for On-Site Source	A-XI-9
A-63	Physical Tests on Concrete Aggregates for Owl Products Company	A-XI-11
A-64	Physical Tests on Concrete Aggregates for 4th Street Crusher	A-XI-13
A-65	Physical Tests on Concrete Aggregates for Beaumont Concrete Company	A-XI-15
A-66	Physical Tests on Concrete Aggregates for C.L. Pharris.	A-XI-16
A-67	Mix Design Requirements for Structural Elements	A-XI-19
A-68	Alluvial Test Fill, Testing Schedule	A-XIII-8
A-69	Rock Test Fill, Testing Schedule	A-XIII-9

Page

Figures

No.	Title	
A-1	Physiography	A-IV-8
A-2	Southern California Tectonics	A-IV-9
A-3	San Andreas Fault Evolution	A-IV-10
A-4	Results of Chemical Tests for Reactivity of Aggregates, On-Site Borrow	A-XI-21
A-5	Results of Chemical Tests for Reactivity of Aggregates, Owl Rock Company	A-XI-22
A-6	Results of Chemical Tests for Reactivity of Aggregates, 4th Street Crusher	A-XI-23
A-7	Results of Chemical Tests for Reactivity of Aggregates, Beaumont Concrete Company	A-XI-24
A-8	Results of Chemical Tests for Reactivity of Aggregates, C.L. Pharris	A-XI-25
A-9	Mortar Bar Reactivity Tests, 4th Street Crusher	A-XI-26
A-10	Mortar Bar Reactivity Tests, Beaumont Concrete Company.	A-XI-27
A-11	Mortar Bar Reactivity Tests, C. L. Pharris	A-XI-28

Plates

No.	Title
A-1	Project Location
A-2	General Vicinity and Borrow Area Map
A-3	Regional Geology
A-4	Local Geology
A-5	Site Geology - Sheet 1
A-6	Site Geology - Sheet 2
A-7	Fault and Earthquake Epicenter Map
A-8	Regional Tectonic Map
A-9	Damsite - Plan of Exploration - Locations of Exploratory Holes
A-10	Damsite - Plan of Exploration - Locations of Seismic Refraction Lines
A-11	Drill Logs - Streambottom - Sheet 1
A-12	Drill Logs - Streambottom - Sheet 2
A-13	Drill Logs - Streambottom - Sheet 3
A-14	Drill Logs - Streambottom - Sheet 4
A-15	Drill Logs - Streambottom - Sheet 5
A-16	Drill Logs - Streambottom - Sheet 6
A-17	Drill Logs - Streambottom - Sheet 7
A-18	Drill Logs - Left Abutment
A-19	Drill Logs - Right Abutment - Sheet 1
A-20	Drill Logs - Right Abutment - Sheet 2

Plates (Continued)

No.	Title
A-21	Drill Logs - Outlet Works - Inlet Portal
A-22	Drill Logs - Outlet Works - Outlet Portal
A-23	Drill Logs - Energy Dissipator
A-24	Drill Logs - Government Canyon Ridge - Sheet 1
A-25	Drill Logs - Government Canyon Ridge - Sheet 2
A-26	Drill Logs - Spillway - Sheet 1
A-27	Drill Logs - Spillway - Sheet 2
A-28	Drill Logs - Spillway - Sheet 3
A-29	Drill Logs - Spillway - Sheet 4
A-30	Rock Quality Parameter Comparisons - Sheet 1
A-31	Rock Quality Parameter Comparisons - Sheet 2
A-32	Rock Quality Parameter Comparisons - Sheet 3
A-33	Rock Quality Parameter Comparisons - Sheet 4
A-34	Rock Quality Parameter Comparisons - Sheet 5
A-35	Rock Quality Parameter Comparisons - Sheet 6
A-36	Seismic Refraction Surveys, Traveltime Curves and Interpretations - Sheet 1
A-37	Seismic Refraction Surveys, Traveltime Curves and Interpretations - Sheet 2
A-38	Seismic Refraction Surveys, Traveltime Curves and Interpretations - Sheet 3
A-39	Seismic Refraction Surveys, Traveltime Curves and Interpretations - Sheet 4
A-40	Downhole Seismic Graphs
A-41	Top of Bedrock Contours
A-42	Embankment - Geologic Profile and Plan
A-43	Embankment - Geologic Sections - Sheet 1
A-44	Embankment - Geologic Sections - Sheet 2
A-45	Government Canyon Ridge, Geologic Profile and Sections
A-46	Spillway - Geologic Sections and Profile
A-47	Outlet Works - Geologic Profile and Subsurface Bedrock Contours
A-48	North Branch Fault Investigations - Location and Logs
A-49	North Branch Fault Investigations - Trench Logs
A-50	Foundation - Soil Logs
A-51	Foundation Materials - Gradation Curves and Adjusted Maximum Dry Densities
A-52	Foundation Materials - Detailed Test Results
A-53	Left Abutment - Soil Logs
A-54	Left Abutment - Results of Laboratory Tests
A-55	Downstream Pervious Borrow Area - Plan of Exploration
A-56	Downstream Pervious Borrow Area - Soil Logs and Gradations
A-57	Downstream Pervious Borrow Area - Gradation Curves
A-58	Downstream Pervious Borrow Area - Maximum Density Tests
A-59	Downstream Pervious Borrow Area - Results of Detailed Laboratory Tests

Plates (Continued)

No.	Title
A-60	Impervious Borrow Area - Plan of Exploration
A-61	Impervious Borrow Area - Fines Distribution
A-62	Impervious Borrow Area - Soil Logs - TH85-1 through TH85-7
A-63	Impervious Borrow Area - Soil Logs - TH85-8 through TH85-14
A-64	Impervious Borrow Area - Soil Logs - TT85-1 through TT85-14
A-65	Impervious Borrow Area - Soil Logs - CBH 86-1 through CBH 86-8
A-66	Impervious Borrow Area - Soil Logs - CBH 86-9 through CBH 86-16
A-67	Impervious Borrow Area - Soil Logs - CBH 86-17 through CBH 86-23
A-68	Impervious Borrow Area - Soil Logs - CBH 86-24 through CBH 86-30
A-69	Impervious Borrow Area - Soil Logs - CBH 86-31 through CBH 86-38
A-70	Impervious Borrow Area - Soil Logs - CBT 86-1 through CBT 86-12
A-71	Impervious Borrow Area - Soil Logs - CBT 86-13 through CBT 86-23
A-72	Impervious Borrow Area - Soil Logs - CBT 86-24 through CBT 86-34
A-73	Impervious Borrow Area - Soil Logs - CBT 87-1 through CBT 87-12
A-74	Impervious Borrow Area - Comparison of In Situ vs. Laboratory Maximum Densities
A-75	Impervious Borrow Area - Results of Detailed Laboratory Tests
A-76	Impervious Borrow Area - Results of Detailed Laboratory Tests (Bentonite Amended)
A-77	Impervious Borrow Area - Stress-Strain Relationships
A-78	Impervious Borrow Area - Moisture-Density Relationships
A-79	Impervious Borrow Area - Moisture-Density Relationships (Bentonite Amended)
A-80	Embankment Section and Details
A-81	Earthquake Accelerograph Records and Response Spectrums
A-82	Embankment Slope Stability Analysis - Design Curves
A-83	Embankment Slope Stability Analysis - End of Construction - Upstream Slope
A-84	Embankment Slope Stability Analysis - End of Construction - Upstream Slope with Earthquake
A-85	Embankment Slope Stability Analysis - End of Construction - Downstream Slope
A-86	Embankment Slope Stability Analysis - End of Construction - Downstream Slope with Earthquake
A-87	Embankment Slope Stability Analysis - Sudden Drawdown
A-88	Embankment Slope Stability Analysis - Partial Pool
A-89	Embankment Slope Stability Analysis - Partial Pool with Earthquake
A-90	Preliminary Analysis of Permanent Embankment Deformations
A-91	Embankment Settlement Analysis
A-92	Embankment Seepage Analysis - Case I - Spillway Crest Elevation (Intact Core)
A-93	Embankment Seepage Analysis - Case II - Spillway Crest Elevation (Cracked Core)
A-94	Embankment Seepage Analysis - Case III - Debris Pool Elevation

Plates (Continued)

No.	Title
A-95	Embankment Seepage Analysis - Transient Seepage Limits
A-96	Concrete Materials Sources - Upper Santa Ana River
A-97	Concrete Aggregate Investigations, Location of Test Trenches
A-98	Gradations for TT87-1 through TT87-4 and Crushed Materials from TT87-3 and TT87-4
A-99	Material Quantities and Distribution
A-100	Embankment Quantities vs. Elevation
A-101	Downstream Pervious Borrow Area - Construction Requirements
A-102	Feature Design Investigations: Test Fills, Test Pits, Test Adits, and Test Excavation Plan
A-103	Abutment Excavation - Profile and Sections

Attachments

No.	Title
A-1	Assessment of Seismic Hazard at the Upper Santa Ana River Damsite by Dr. Clarence Allen
A-2	Seismological Report for Seven Oaks Dam for Santa Ana River Basin, California by Bruce A. Bolt
A-3	Woodward-Clyde Consultants, Logs of Borings and Reports on Aquifer Test and Geophysical Investigations
A-4	Leighton and Association Geophysical Report
A-5	South Pacific Division Laboratory Report of Soil Tests

I. INTRODUCTION

Purpose and Scope

1-01 Geotechnical investigations were conducted to determine the extent, distribution, and physical properties of the rock and soils at the proposed Seven Oaks damsite, and to obtain detailed information on the foundation, required excavations, construction materials and groundwater conditions. This appendix discusses the geologic and seismologic conditions at the site and describes the geotechnical investigations and testing conducted for the project. This appendix also describes the methods of analysis used, summarizes the foundation conditions, and recommends design values, foundation treatments, embankment, spillway and outlet works designs and construction procedures. Data and recommendations are of sufficient detail to lead to a test excavation and test fill construction contract, an abutment excavation contract, and to the FDM design phase for the embankment, spillway and outlet works.

Organization of Appendix

1-02 After a description of the geology, groundwater and seismicity of the site, the appendix is organized basically to discuss what investigations and testing were conducted, the results of those investigations and tests, and the analysis of those results. These topics are discussed for each major project feature in sections V through VIII. Based upon the analysis of the investigation results, design values for the foundation and embankment materials are recommended in section IX. These design values are in turn used to develop a project design which is presented in section X along with the required analysis of that design. Sections XI and XII deal with construction materials, procedures and requirements. Section XIII proposes a construction schedule. Quality control and instrumentation requirements are presented in sections XIV and XV. It is recognized that this organizational format is somewhat cumbersome and necessarily

repetitive, however, the intent is that each section and the discussion of each project feature can be read more or less independently of each other.

Project Features

1-03 Seven Oaks Dam would be a zoned earth and rockfill embankment with a maximum height above the existing streambed of approximately 550 feet. After excavation of the streambed alluvium in the canyon and the preparation of the bedrock foundation, however, the maximum embankment height would exceed 650 feet. The purpose of the dam would be flood control and debris storage only. The embankment would be arched upstream and would include an impervious central core, wide filter and transition zones and free-draining rockfill shells with 1V on 2H outer slopes both upstream and downstream. The dam would retain floodflows on the Santa Ana River and its mountain tributaries with a gross flood storage capacity of 145,600 acre-feet at spillway crest. A concrete-lined outlet tunnel through the left (east) abutment would be used during construction of the embankment for diversion of groundwater and the Santa Ana River, and subsequently to control releases of floodwaters from behind the completed structure. A detached, unlined 500-foot wide spillway would be excavated through a saddle on the east side of the canyon. Spillway flows would be discharged into Deep Creek which, in turn, empties into the Santa Ana River several thousand feet downstream from the damsite. A maximum cut in rock of 375 feet would be required at this spillway site. An additional excavation would be required through a ridge immediately upstream from the embankment to provide necessary drainage from a tributary canyon to the outlet works.

Location

1-04 The damsite is located about 1 mile upstream from the mouth of Santa Ana River Canyon, and about 4 miles north-northeast from the town of Mentone, California, (north half of section 4, T1S, R2W, SBB). The general location of the project is shown on plate A-1. A more detailed vicinity map showing the embankment, spillway, and borrow areas is presented on plate A-2. The reservoir at full pool conditions would extend approximately 3 miles upstream in Santa Ana River Canyon and would also inundate portions of Government Canyon and Warm Springs Canyon. The damsite and reservoir are almost entirely within the boundaries of the San Bernardino National Forest in San Bernardino County. The approximate existing elevation of the streambed at the damsite is 2,060 feet National Geodetic Vertical Datum of 1929 (NGVD).

II. GEOLOGY

Regional Geologic Setting

2-01 Upper Santa Ana River Canyon is within the southern frontal margin of the San Bernardino Mountains, which are in the eastern part of the Transverse Ranges physiographic province in Southern California (fig. A-1). The Transverse Ranges province is an elongated geomorphic and structural unit that trends essentially east-west, and is made up of chains of mostly parallel mountain ranges and valleys extending from Point Arguello eastward to the southern Mojave Desert. The principal geomorphic and structural features of the Transverse Ranges lie across the grain of adjacent physiographic provinces, which are strongly influenced by the northwest-southeast-trending San Andreas fault system. Within and bounding the Transverse Ranges province are other major fault zones that have been active during the same span of geologic time that the San Andreas system has been active. Rock units within the province are represented by rocks that range from Precambrian plutonic and metamorphic types to Cenozoic sedimentary rocks. Complex sections of Cretaceous plutonic rocks dominate the province.

RECENT GEOLOGIC HISTORY AND TECTONICS

2-02 Southern California is a tectonically active region, experiencing crustal compression, dextral shear, thrusting, block rotation, and allied folding and faulting. The present tectonic regime is usually modeled with the portion of Southern California southwest of the San Andreas fault attached to the Pacific plate and moving northwesterly relative to the North American plate (fig. A-2). This strike-slip action has also caused dextral (right-lateral) shear in the crustal material on both sides of the fault, forcing blocks of crustal material to rotate clockwise (Luyendyk and others, 1980). In addition, extension of the Basin and Range has added a component of force from the northeast which has forced the San Bernardino Mountains to thrust up against the San Jacinto Mountains at San Gorgonio Pass. Deep seismic activity has been identified in a zone 12-25 miles southeast of the damsite, and this activity has been interpreted to be due to a north-dipping thrust acting

at depth (Nicholson and others, 1986). Studies of subsurface temperature and density indicate that subcrustal material is upwelling beneath the Salton Trough and downwelling beneath the Transverse Ranges (Humphreys, 1986). The downwelling action may be driving the north-dipping thrust at depth indicated by the deep seismic activity southeast of the damsite. In summary, the current geological situation in the region includes not only the faulting, fluvial erosion, and fluvial deposition observed on the surface, but also relative movement, rotation, and thrusting of crustal material as well as downwelling of subcrustal material, all occurring simultaneously.

SAN ANDREAS FAULT SYSTEM

2-03 The San Andreas fault system is the master transform along which the Pacific plate moves northwesterly relative to the North American plate. In Southern California, the fault strikes northwesterly from the Imperial Valley to the Coachella Valley, then more west-northwest through the Transverse Ranges to the southwest corner of the San Joaquin Valley, and then northwesterly again into Northern California (fig. A-2). The region where the fault curves from WNW to NW is known as the "Big Bend." Since initiation of movement along the fault in the Miocene epoch, cumulative right-lateral offset along this system has been anywhere from about 100 miles (Frizzell and others, 1986) to about 180 miles (Luyendyk, 1987). Over this time, slip rates have varied from about 1/2 to 2 in/year, depending upon location. These rates have characterized the fault system for the past few million years. The part of the San Andreas fault that passes through the Transverse Ranges province of Southern California is its most complex segment. In this region, the San Andreas fault and several others constitute a complex system of anastomosing faults within a strip as wide as 6 miles.

2-04 In the San Bernardino area from Cajon Pass southeastward, the San Andreas fault forms the southwest-facing margin of the San Bernardino Mountains to a point north of San Bernardino where it splits into two branches, the North Branch and the South Branch (Dibblee, 1964) (pl. A-8). The North Branch aligns with the upper, linear portion of Mill Creek Canyon and then curves southeastward into the Coachella Valley. The South Branch continues along the face of the San Bernardino Mountains to a point northeast of Yucaipa where its surface expression is less obvious and is thought to curve southward to the Banning fault. The Banning fault, also a named segment of the San Andreas system, trends eastward and then southeastward, eventually joining the North Branch of the San Andreas fault in the Coachella Valley.

2-05 The complexity of the fault system in this area is due to the complicated evolution of the fault combined with the aforementioned crustal thrusting. Although the scientific community has not agreed upon the details of the tectonic evolution, it will be described using the model set forth by Matti and others (1985). As used in their model, three faults, the Wilson Creek, Mill Creek, and Mission Creek were proto-San Andreas faults, each taking its turn in the tectonic evolution

of the San Andreas (fig. A-3). The Mill Creek and eastern portion of the Mission Creek faults are equivalent to the North Branch of the San Andreas.

2-06 First, displacement along the Wilson Creek fault juxtaposed the San Bernardino Mountains against the material which is now between the North and South Branches of the San Andreas fault, which will herein be referred to as the Wilson Creek block and is the location of the Seven Oaks damsite (fig. A-3-A). Second, due to left-slip on the Pinto Mountain fault obstructing movement on the Wilson Creek fault, right-lateral displacement was transferred to a new fault, the Mission Creek fault, which evolved to the south of the Wilson Creek fault (fig. A-3-B). This juxtaposed material similar to that in the San Gabriel Mountains (SGMB) against the San Bernardino Mountains and the Wilson Creek block, which thereafter behaved as a single structural unit (fig. A-3-C). Third, continued left-slip on the Pinto Mountain fault obstructed movement on the Mission Creek fault (fig. A-3-D), and right-lateral displacement was transferred to yet another new fault, the Mill Creek fault, which evolved north of the Mission Creek and Wilson Creek faults (fig. A-3-E). Only about 5 miles of right-lateral displacement was generated by the Mill Creek fault prior to its obstruction by the continued left-slip on the Pinto Mountain fault. Concurrently, right-slip on the San Jacinto fault further separated the San Gabriel Mountains from its related material left behind at the southern edge of the San Bernardino Mountains (fig. A-3-E). Thrusting of the San Bernardino Mountains southward has further complicated the geologic structure, and apparently transferred the primary right-lateral movement from the throughgoing North Branch of the San Andreas to the combination of the Banning fault and the South Branch of the San Andreas, which probably reactivated the western portion of the Mission Creek fault (fig. A-3-F).

SAN BERNARDINO MOUNTAINS

2-07 The San Bernardino Mountains have been uplifted since the late Pliocene along northern frontal faults and the San Andreas and Pinto Mountain faults (Dibblee, 1975). They consist of steep-walled deep canyons, a subdued and markedly discontinuous upland surface, and several ridges and peaks including San Gorgonio Peak (elevation 11,502 feet), the highest point in the Transverse Ranges. The northern margin of the range is represented by a north-facing erosional escarpment and south-dipping thrust faults (Dibblee, 1975). The eastern margin of the range dips more gradually into Mojave Desert sediments. To the west, the San Bernardino Mountains are separated from the San Gabriel Mountains by the San Andreas fault near Cajon Pass. The steep, southwest-facing margin of the range aligns with the trace of the San Andreas fault and its South Branch to a point northeast of Yucaipa where the southern margin is then considered to resume about 5 miles to the south and extend eastward along the Banning fault from Cherry Valley to the west end of the Coachella Valley (pl. A-8).

2-08 Geologically, the portion of the San Bernardino Mountains lying south of the North Branch of the San Andreas fault, the Wilson Creek block, contains different rock units than the portion north of the North Branch (Dibblee, 1975). The rocks to the north are similar to basement rocks of the Mojave Desert crustal block. However, the rocks to the south are similar to basement rocks of the San Gabriel Mountains to the northwest. A cumulative list of rock units exposed within the San Bernardino Mountains (on both sides of the North Branch) would include Precambrian plutonic and gneissic rocks, Paleozoic quartzites, limestone, dolomite, and marble, Mesozoic plutonic rocks, Tertiary terrestrial clastic rocks and basalt, and Quaternary alluvial and colluvial deposits.

Local Geology and Topography

TOPOGRAPHY

2-09 The proposed damsite and reservoir area is within the steep-walled Santa Ana River Canyon along the southern margin of the San Bernardino Mountains. The elevation of the canyon floor at the proposed damsite is approximately 2,060 feet (NGVD). The gradient of the canyon floor averages about 3 percent. Elevations of the ridgetops outlining the canyon range from about 3,500 feet directly above the damsite on the left (east) abutment, to almost 4,000 feet on the western flank of the canyon. The canyon walls at the right (west) abutment are very steep 1V on 0.6H just above the canyon floor and then flatten to 1V on 2H at higher elevations. The "ridge and swale" topography at the left abutment has more uniform slopes of about 1V on 1.5H. Several "hanging valleys," probably created by the rapid uplift of the San Bernardino Mountains and equally rapid down-cutting by the Santa Ana River, can be identified nearby the site.

ROCK UNITS

2-10 Rock units exposed within the damsite and reservoir area consist of Precambrian quartzo-feldspathic gneiss with biotite, Cretaceous (?) biotite-rich quartz diorite and porphyritic quartz monzonite, Tertiary Mill Creek formation, and Quaternary terrace/alluvial deposits. Rock unit identifications are based upon correlations between observations made during field reconnaissance and geologic mapping and published literature (Dibblee, 1968, 1974; West, 1987).

Gneissic Rocks (gn)

2-11 The oldest recognizable rock terrane is a Precambrian (?) quartzo-feldspathic gneiss that has been intruded by quartz diorite (qd). These older rocks are typically layered with bands and lenses of quartzite, quartzo-feldspathic gneiss, and occasional dark schistose layers. In addition to the intrusion by the diorite, they are transected by dikes of aplite, pegmatite, lamprophyre, amphibolite and fine-grained basaltic to rhyolitic rocks. These rocks display a strong mineralogical

and textural foliation that trends generally northeasterly to easterly, and dips steeply to the south. They are often highly sheared, with planar orientations trending essentially west-to-northwest and dipping at various steep angles from vertical. This gneissic rock terrane occurs as a fault-bounded wedge between the North and South Branches of the San Andreas fault.

Quartz Diorite (qd)

2-12 Intruding the older gneissic rocks (gn) is a biotite-rich, quartz diorite of Cretaceous age (Dibblee, 1968). These rocks are moderately to strongly jointed and fractured. They are also slightly foliated. Foliation trends generally easterly and dips steeply to the north and south. Exposures of these rocks, along with the gneissic rocks (gn), occur only within a fault-bounded wedge between the Northern and Southern Branches of the San Andreas fault. Areas on published geologic maps (Dibblee, 1974) mapped as quartz diorite (qd) also contain large inclusions of roof pendants of gneissic rocks (gn). These inclusions were not differentiated during the large scale field mapping conducted as a part of this study, but were identified during the detailed mapping at the damsite.

Quartz Monzonite (qm, qmp)

2-13 Quartz monzonite porphyry (qmp) and quartz monzonite (qm) of Cretaceous age occur only north of the North Branch of the San Andreas fault within the reservoir area. The quartz monzonite (qm) is mostly equigranular and contains occasional xenoliths of mafic minerals. Although it lacks foliation, it is generally moderately jointed with most joints dipping near vertical. The quartz monzonite porphyry (qmp) contains large phenocrysts of feldspar, up to 7 cm in diameter, in a matrix of fine-grained feldspar and quartz. Collectively, these rocks form bold cliff faces along the flanks of Santa Ana River Canyon and the steep topography of the adjacent highlands (Schaefer Dixon Assoc., 1985).

Tertiary Sediments (Tmc)

2-14 Within the project area, the Mill Creek formation (Tmc) of Tertiary age is exposed only as fault slivers between the major splays of the North Branch of the San Andreas fault zone in Warm Springs Canyon, on Government Canyon Ridge and in Government Canyon. Exposures of these rocks consist of highly folded and faulted, terrestrial and lacustrine sandstone, shale and conglomerate. The Mill Creek formation was probably deposited in a pull-apart basin associated with early movements of the San Andreas fault (West, 1987).

ALLUVIAL/COLLUVIAL UNITS

Terrace Deposits (Qt)

2-15 Dissected remnants of at least three generations of Quaternary-age stream terrace deposits (Qt) were identified in Santa Ana River Canyon during the field mapping. At the damsite, the terrace deposits occur

as patches of sand, gravel, cobbles, and boulders "hanging" on the walls of the steep-sided Santa Ana River canyon. On Government Canyon Ridge, the terraces are more extensive in area. Each generation was identified based upon the elevation of the base of the deposit. No published data was found describing these deposits in detail. According to Schaefer Dixon Associates (1985), these terraces have probably formed in a similar manner to those terrace deposits in the adjacent San Gabriel Mountains that are the result of both tectonic base-level changes and climatic perturbations. The terraces probably developed during the late Quaternary time when conditions were much wetter, and both the San Gabriel and San Bernardino Mountains were being uplifted. Field work will continue through the FDM study phase to determine the ages, regional extent and character of the terraces. Preliminary results indicate the ages of the three terraces mapped within the proposed damsite area to be 12,000 (Qt3), 55,000 (Qt2) and 500,000 years old (Qt1) (Weldon, written comm.).

Colluvium (Qc)

2-16 Colluvium (Qc) consists of materials deposited by soil creep, slope-wash, and gravity (talus). Generally, the larger of these deposits are estimated to range from 5 to 10 feet in thickness on the upper slopes to approximately 40 to 50 feet in thickness on the lower slopes. A localized deposit of colluvium at the left abutment is at least 116 feet thick.

Landslide Debris

2-17 Landslide debris in the vicinity of the damsite is identified mainly on the basis of topographic expression. Suspect areas were delineated during the detailed aerial photo analysis and later field checked where possible. In the rock terrane north of the North Branch, the areas designated as landslides are predominantly rock falls. South of the fault, in the immediate vicinity of the damsite, the landslide deposits are more characteristically debris flows. In Warm Springs Canyon, landslide masses are typically associated with the thick sections of Tertiary sedimentary rocks (Tmc) which outcrop between the splays of the North Branch of the San Andreas fault (Schaefer Dixon Associates, 1985). See section VIII of this appendix, for a further discussion of landslides, especially as they affect the reservoir area.

Stream Terrace (Qst)

2-18 Stream terrace material is composed of streambed alluvium deposited during high flood activity. These materials consist of sand, gravel, cobbles and boulders, similar to Qal, but shows sand stability by surficial cover of fine-grained sediments and vegetation. These interfinger with colluvium (Qc) along edges of the canyons.

Recent Alluvium (Qal)

2-19 Santa Ana River Canyon is floored by clastic materials eroded from the adjacent highlands and transported down the canyon during various stages of flooding. These materials consist of unconsolidated sand,

gravel, cobble, and boulder size clasts randomly deposited within the 500- to 1000-foot wide main channel of the canyon. Some boulders are the size of a small truck.

FAULTING

2-20 Within the immediate damsite vicinity, the most prominent structural feature is fault "A", shown on plates A-4 and A-6. It is located directly south of the damsite near the outlet portal. This steeply dipping fault juxtaposes gneiss (gn) and diorite (qd) on each side of Santa Ana River Canyon. However, away from the canyon bottom, the nature of the fault contact is indistinct and it appears to widen into a vague zone of mixed gneiss and diorite. Although fault "A" could represent a splay of the North Branch of the San Andreas fault, it is more likely a structure unrelated to the present tectonic regime. It is overlain by unfaulted terrace deposits (Qt2) estimated to be approximately 55,000 years old (Weldon, written comm.) and is apparently offset slightly by minor low angle faults on the west side of the canyon. Further to the west, it can not be traced to the Alder Creek Road cut, less than one mile away, and on the east side there is no evidence from field mapping or aerial-photo lineament analysis to suggest that it joins with the San Andreas fault.

2-21 Other fault and shear zones within the site vicinity could only be observed for short distances before they were hidden by colluvium or otherwise terminated. Most of these faults trend west to north west and dip steeply to the north and south. One diorite block, south of the proposed damsite appears to be fault-bounded with the gneiss. None of these faults are known to be active.

III. GROUNDWATER

Surface Flow

3-01 Streamflow is perennial in the mountainous canyons of the Santa Ana River and intermittent downstream from the mouth of the canyon where low flows percolate into the Bunker Hill-San Timoteo groundwater basins. The natural flow in the upper reaches of Santa Ana River Canyon is partially regulated by releases from Big Bear Lake. The major part of this regulated flow, and unregulated flows on the Santa Ana River itself, is diverted for power generation and water supply near the confluence of Bear Creek and the Santa Ana River. In spite of these diversions, there is normally some surface flow at the damsite. During the summer/fall of 1987 after an especially dry winter, there was some surface flow in the area of the proposed upstream toe of Seven Oaks Dam, while at the centerline the riverbed was dry. Periodically, the Santa Ana River and its tributaries experience severe flooding. The flood hydrographs are characterized by sharp peaks of short duration. Peak flows in excess of 50,000 ft^3/sec have been measured on the Santa Ana River near Mentone.

Subsurface Flow

3-02 Subsurface as well as surface flow from Santa Ana River Canyon enters the Bunker Hill-San Timoteo Basin downstream from the canyon mouth. A California Department of Water Resources study (1970) estimated 2,500 acre feet/year for the amount of subsurface outflow from Santa Ana River Canyon. This estimate assumed an average permeability of 1,000 gal/day, a hydraulic gradient of 0.03 ft/ft and a 75,000-square foot cross-sectional wetted area. At the damsite, approximately 1 mile upstream from the canyon mouth, the permeability of the alluvium was calculated from aquifer test data to be 2,200 gal/day and the saturated cross-sectional area of the alluvium is approximately 30,000 square feet. The hydraulic gradient is 0.03ft/ft. Using these variables, the estimated outflow is calculated to be approximately 2,200 acre-feet/year.

Neither estimate, however, considers the possible barrier effects of the South Branch of the San Andreas fault at the canyon mouth where the fault trace is partially defined by a vegetation lineament which is indicative of differential groundwater conditions.

Groundwater Levels

3-03 The damsite and reservoir are in the Santa Ana River Canyon hydrologic subarea. Within the subarea, the closest water wells are downstream from the mouth of the canyon (T1S/R2W 8C01) and upstream in the middle of the proposed reservoir near Southern California Edison powerhouse No. 2 (Bear Valley Well No. 2). At the damsite, groundwater data was obtained from five observation wells installed during exploratory drilling in 1984, 1986 and 1987. Observation well R86-40 is located near the proposed upstream toe while the remaining four are located near the dam centerline. See plate A-9 for the exact locations. The results from these wells are shown in table A-1.

Table A-1. Groundwater Levels in Observation Wells.

Well: Date of Measurement	R84-01	R84-02	R84-03	R86-40	C86-35
	Depth to Groundwater Below Ground Surface (feet)				
Jun '84	6.0	8.4	10.3		
Jun '85		8.7	10.1		
Sep '85			9.8		
May '87	5.4	8.6	10.0	10.1	10.8
Sep '87	9.6	13.0		13.1	17.2
Mar '88		8.6		9.9	11.3

3-04 Groundwater levels downstream of the canyon mouth have been monitored by the San Bernardino Valley Water Conservation District. The depth to groundwater in three of the monitoring wells located near the proposed downstream pervious borrow area is discussed in paragraph 12-38 of this appendix.

IV. SEISMICITY

Regional Seismicity

4-01 The project is located within Zone 4 of the Seismic Risk Zone Map (EM 1110-2-1902, change 1, 1982). The region surrounding the Seven Oaks damsite is seismically active due to the tectonic interaction of the North American plate and the Pacific plate. Relative movement along the boundary between the plates has formed the San Andreas fault system, with the action of movement being primarily right-lateral strike-slip. In addition, the regional shear field developed by the plate interaction has caused crustal breakage in the form of additional strike-slip faults, dip-slip faults, oblique-slip (exhibiting both strike-slip and dip-slip) faults, and rotation of crustal blocks between faults. The relative movement between the crustal plates as well as the movements within the plates cause release of energy in the form of earthquakes. The magnitude and recurrence interval of the maximum credible earthquake associated with any given fault is variable and dependent upon the postulated length of fault rupture, orientation of movement (strike-slip, normal, thrust, etc.), and measured or inferred patterns of events. The San Andreas fault is expected to rupture with the largest maximum credible event in the region: an earthquake of Richter magnitude greater than 8.

San Andreas Fault System

4-02 Seismic activity along the San Andreas fault system suggests that faulting related to this zone of crustal movement extends more than 700 miles, from the Gulf of California to the Pacific Ocean north of San Francisco. The seismic behavior of the San Andreas fault system varies markedly along its length. Three distinct modes of behavior are apparent: (1) moderately frequent large shocks together with significant seismic creep along the Coachella Valley-Salton Trough segment and the Mendocino fracture zone, (2) frequent small to moderate shocks along the segment from Hollister to the Big Bend, and (3) infrequent great earthquakes (e.g., 1857 and 1906) along the northern segment (Cape Mendocino to Hollister) and Transverse Ranges segment (Big Bend to Coachella Valley).

4-03 A geologic study of prehistoric earthquakes on the San Andreas fault at Pallet Creek (approximately 35 miles northwest of the damsite) indicated that since 500 A.D. six earthquakes comparable in size to the great 1857 Fort Tejon earthquake (M 7.9) have occurred (Sieh, 1978). The mean recurrence interval of those earthquakes was calculated to be about 150 to 200 years. Surface-rupture features associated with the 1857 earthquake, which was centered approximately 115 miles from the Seven Oaks damsite, were reported to have extended to a point approximately 30 miles northwest of the site. According to various earthquake recurrence models prepared by Weldon and Sieh (1985), the next major event on the San Andreas fault in Southern California will likely be centered in the vicinity of the eastern Transverse Ranges.

Other Major Faults

4-04 As shown on plate A-7, the Fault and Earthquake Epicenter Map for the Seven Oaks damsite, numerous other faults exist within approximately a 50-mile radius from the site. These include the San Jacinto fault and Whittier-Elsinore fault, each of which is capable of generating an earthquake in the magnitude range of 7 to 7.5. However, the greater distance from the damsite and smaller magnitude potential of these faults leave the San Andreas fault as the causative fault for the design earthquake (M 8+) at the site.

Subsidiary Faulting

4-05 Secondary faulting associated with the design event on the South Branch of the San Andreas fault can probably be expected. Sympathetic movement on random shears within bedrock is likely; however, the amounts of displacement are speculative. In an assessment of the fault rupture hazard at the Seven Oaks damsite, Clarence Allen recommended that a design displacement of as much as 4 feet in any direction at the damsite from a magnitude 8+ earthquake on the San Andreas fault should be assumed (see attachment A-1).

4-06 Further uncertainties exist as to the possibility of sympathetic movement on the North Branch of the San Andreas fault. The most recent investigations on the North Branch fault zone, as described in section V of this appendix, indicate that a late Pleistocene terrace (estimated to be about 55,000 years old) has been offset at least several feet. This equates to a slip rate of a few tenths of a millimeter per year, which is about two orders of magnitude less than the slip rate on the South Branch (Weldon, written comm.). These offsets could be the result of continuing tectonic activity along the North Branch or the result of sympathetic movement related to a major earthquake, or earthquakes, on the South Branch. Based upon the apparent minor amount of total offset on the North Branch zone over the last few tens of thousands of years and its close proximity to the South Branch, which is postulated to have experienced numerous large earthquakes, it is believed that sympathetic movement on the North Branch due to the project design earthquake will be insignificant.

Earthquake History

4-07 The earthquake history of the Seven Oaks damsite is dominated by the San Andreas fault system. As previously noted, the San Andreas system can be separated into segments based upon its seismic behavior. The damsite lies within the Transverse Ranges segment between the Big Bend and the Coachella Valley. This segment is subjected to conflicting tectonic forces, east-west extension and north-south compression. The apparent result of this interaction between tectonic forces is a section of the fault which is relatively quiet seismically when compared to activity to the north and south of the Transverse Ranges segment. Within this segment, the San Andreas fault zone locally changes from a single narrow strand exhibiting predominantely right lateral offsets to a zone of splayed faulting which includes a component of reverse or reverse-oblique movement on north dipping fault planes. This tends to produce a more diffuse seismic pattern than exhibited within the other segments. Seismic events tend to form short arcuate patterns, tight clusters, or short east-west trends rather than long linear patterns common to strike-slip movement along near vertical fault planes. Closer examination and reevaluation of the mechanisms involved in the faulting between the Big Bend and the Coachella Valley indicate that simple interaction between plate boundries is too simplistic an explanation to account for the difference in seismic expression along this segment.

4-08 Instrumentally recorded seismicity in the vicinity of the damsite is comparatively low. There has been an apparent increase in low level activity since about 1970, however, it is believed to be the result of both increased instrumentation and improved monitoring techniques rather than an actual change in seismic activity. Between 1935 and 1986, three earthquakes greater than Richter magnitude 4 occurred within 10 miles of the site. These were a M 4.6 on 19 February 1940 (7.5 miles), a M 4.0 on 16 October 1951 (6.6 miles), and a M 4.3 on November 1978 (6.8 miles). The closest event of magnitude 5 or greater was a M 5.5 on 29 August 1943 approximately 12 miles from the site. The largest pre-instrumental historic earthquakes within 25 miles of the site were magnitudes 6.0 (1923), 6.7 (1899), and 6.8 (1918). Even though the historic seismicity near the site is relatively low as compared to the San Andreas fault zone in general, the seismic potential remains high due to the apparent periodicity of major earthquakes along this segment of the San Andreas fault (Sieh, 1978). See plate A-7 for a plot of instrumentally recorded earthquakes within a 50-mile radius of the damsite as well as historic events greater than magnitude 6.

Seismic Probability

4-09 The great 1857 Fort Tejon earthquake (magnitude 7.9) was the last major earthquake along the San Andreas fault which would have affected the damsite. Major slip on the segment of the fault adjacent to the damsite could also have occurred in association with the earthquake of 8 December 1812, (Jacoby and other, 1987). Because of the short historical record and the infrequent nature of large seismic events,

recurrence intervals are estimated to a large extent using data from paleoseismological studies. Such studies conducted by Kerry Sieh (1978) in the Cajon Pass and Salton Sea areas indicate recurrency intervals of approximately 130 and 220 years, respectively.

4-10 The United States Geological Survey (USGS), following a recommendation of the National Earthquake Prediction Evaluation Council (NEPEC), formed the Working Group on California Earthquake Probabilities to evaluate the probability of the occurrence of a large (magnitude 7 or greater) earthquake in California. Their results were published in USGS Open-File Report 88-398. In this report, the Working Group recognized the uncertainty of the behavior of the San Andreas fault in Southern California which they divided into 3 segments; the Mojave and San Bernardino Mountains segments (which correspond generally with the previously described Transverse Ranges segment) and the Coachella Valley segment. Their conclusion is that within the next 30 years there is a 30, 20, and 40 percent probability, respectively, of a large earthquake being generated on these 3 segments individually and a combined probability of 70 percent for a large earthquake in Southern California.

Earthquake Ground Motion

4-11 The maximum credible earthquake at the Seven Oaks damsite is a magnitude 8+ event along the South Branch of the San Andreas fault. For conservatism, the main release of seismic energy from such an earthquake is assumed to occur at the mouth of the canyon, less than 1 mile from the damsite. Realistically, however, the predicted great earthquake could occur anywhere along the "locked" Transverse Ranges segment of the fault. Lateral displacement along the fault plane itself would likely be on the order of 20 feet and subsidiary displacements at the damsite are estimated to be as much as 4 feet in any direction. In attachment A-2, Bruce Bolt (1987) estimates the (mean) ground motion parameters on rock at the site for both a maximum credible event within 2 km (1.2 mi) of the site and a maximum probable event within 20 km (12 mi) of the site. These values, taken from table 1 of attachment A-2, are listed in table A-2.

Table A-2. Estimated (Mean) Ground Motion Parameters on Rock at Site.

	Max. Credible Event (Adjacent San Andreas)	Max. Probable Event (in 50 years)
Source Distance to Damsite (km)	2	20
Magnitude (Ms)	8+	7.5 - 8.0
Seismic Moment (dyne-cm)	$10^{(28)}$	$10^{(27)}$
Recurrence Rate	150 +/- 30 years	1 percent chance per year
Peak Horizontal Acceleration	0.70g	0.5g
Peak Horizontal Velocity (cm/sec)	90-105	70-80
Peak Horizontal Displacement (cm)	50-80	40-70
Bracketed Duration at 0.05g (sec)	40	35
Predominant Period in ground Velocity (sec)	0.2-10.0	0.3-8.0

Reservoir-Induced Seismicty

GENERAL

4-12 Reservoir-induced seismicity (RIS) is defined as the occurrence of earthquakes which are triggered by or through the operation of a reservoir (Meade, 1982). Since there are no physical differences between a naturally occurring earthquake and one that has been induced by reservoir loading, the probability of an event having been reservoir induced is determined by a statistical comparison of deviations between the pre- and post-impoundment seismic patterns or histories. The major types of evidence commonly used to analyze whether or not an event or sequence of events were reservoir-induced, in descending order of importance, are:

a. Post-impoundment increase in seismicity,

b. Correlation of water level (or similar reservoir parameter) and seismic activity, and

c. Display of a distinctive aftershock pattern as expressed by an unusually high slope of the magnitude-frequency relationship (b-value evidence) (Meade, 1982).

4-13 A study of reservoirs throughout the world which have experienced RIS appear to have certain features in common (Packer, et. al., 1979). The most important of these factors are:

a. The presence of a relatively permanent deep reservoir (greater than 300 feet),

b. A site location that is in a tectonically prestressed condition, and

c. The presence of active and/or potentially active faults in or near the reservoir area. Potentially active faults are faults which are capable of movement within the current stress field.

It should also be noted that RIS events have occurred when one or more of these factors have been absent while no change or even a decrease in seismic activity has resulted even though all three parameters were present.

SEVEN OAKS DAMSITE

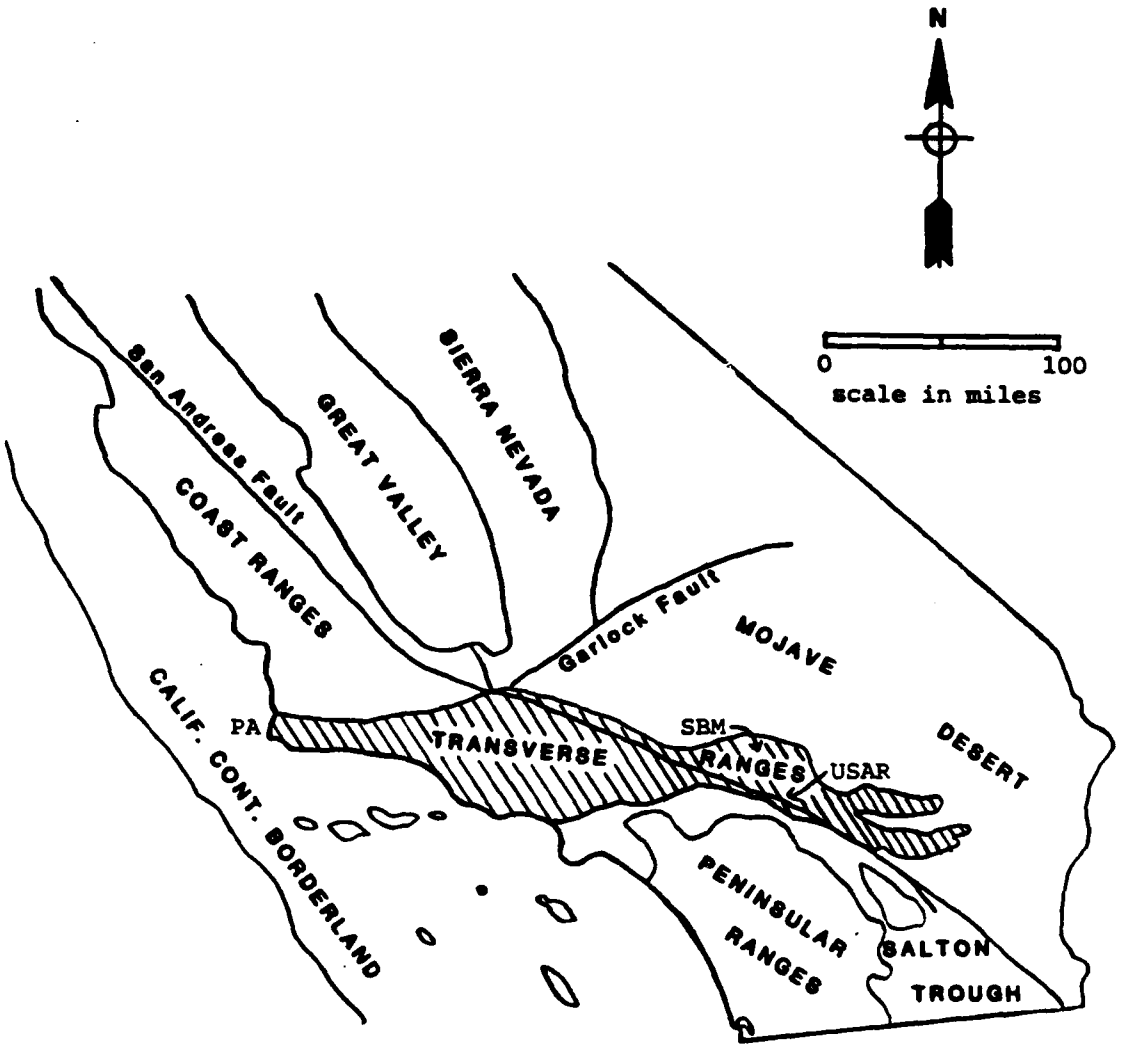
4-14 The depth of water in the reservoir is the main factor for analyzing RIS rather than the actual areal extent of the reservoir. Relatively permanent deep reservoirs (greater than 300 feet) appear to be more susceptible to RIS. The strongest evidence in support of RIS activity has been a close temporal association between reservoir filling (loading) followed by an increase in seismic events. The apparent response time may vary from a few days to a few years but the relationship is most obvious when the lag time between cause and event is from a few weeks to a few months. The strongest correlations tend to occur in dams with deep permanent reservoirs which are subjected to water fluctuations (cyclic loading). Even though the Seven Oaks dam will be subjected to cyclic loading, hydrologic filling frequency curves indicate that it will only contain enough water to classify it as a deep reservoir (300 foot water depth immediately behind the dam) statistically once every 65 years.

4-15 The Seven Oaks damsite is located in a tectonically prestressed area. In certain cases of suspected RIS, reservoir loading appears to have acted as a triggering mechanism, releasing stored strain energy in areas where the crustal blocks are already strained to near-yield conditions. The largest event to date that has been associated with reservoir loading has been a magnitude 6.5 at Koyna, India, in 1967.

4-16 Ruptures produced by RIS events have occurred only along prestressed, tectonically active or potentially active faults (Packer, et. al., 1979). Tectonically active faults are defined as faults which were formed during the current tectonic regime. Of the two significant faults in the immediate vicinity of the dam, the South Branch of the San Andreas lies downstream of the proposed embankment while the North Branch fault zone transects the potential deep reservoir area.

Because of its location, the North Branch would have the greater potential for RIS loci. However, even though there appears to have been movement on the North Branch within the last 55,000 years, the apparent offsets are at least one and possibly two orders of magnitude less than for the South Branch. Indeed, the offsets observed may be related to movement sympathetic to activity of the South Branch, rather than to independent earthquakes on the North Branch itself.

4-17 Because the Seven Oaks damsite is located in a tectonically prestressed area with active or potentially active faults in or near the reservoir area and because the dam will occasionally impound a reservoir more than 300 feet deep, it has characteristics which have been associated with RIS. However, it may not be possible to determine whether or not any given event is due solely to reservoir loading or whether reservoir loading was a contributing factor. To aid in that determination, a study to determine the pre-impoundment seismic pattern of the damsite is in progress. An additional tri-component recording station will be installed at or near the damsite during the Embankment FDM study phase to augment the existing seismic network in the area.



Abbreviations: PA = Point Arguello; SBM = San Bernardino Mountains; USAR = Upper Santa Ana River. (After Dibblee, 1982.)

SANTA ANA RIVER BASIN, CALIFORNIA
SANTA ANA RIVER MAIN STEM

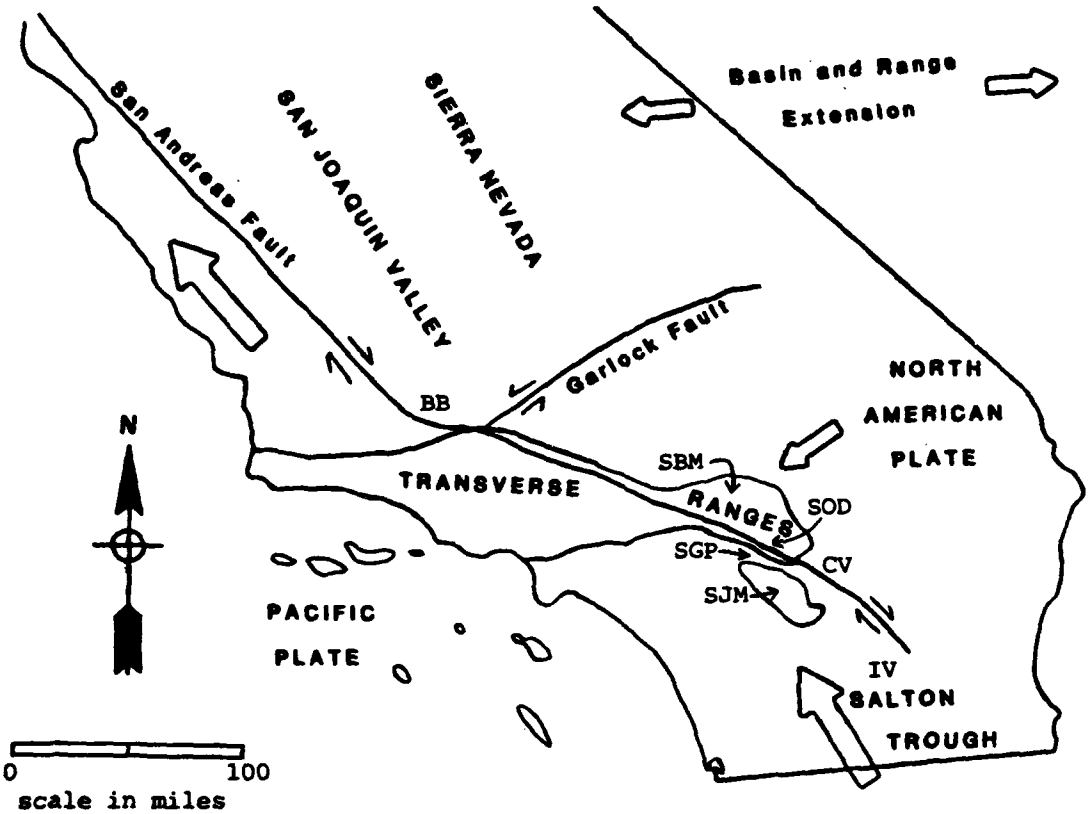
PHASE II - GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

Physiography

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

FIGURE A-1



Abbreviations: BB = Big Bend; CV = Coachella Valley; IV = Imperial Valley; SBM = San Bernardino Mountains; SGP = San Gorgonio Pass; SJM = San Jacinto Mountains; SOD = Seven Oaks Dam (project location). Arrows represent relative crustal motions.

SANTA ANA RIVER BASIN, CALIFORNIA
SANTA ANA RIVER MAIN STEM

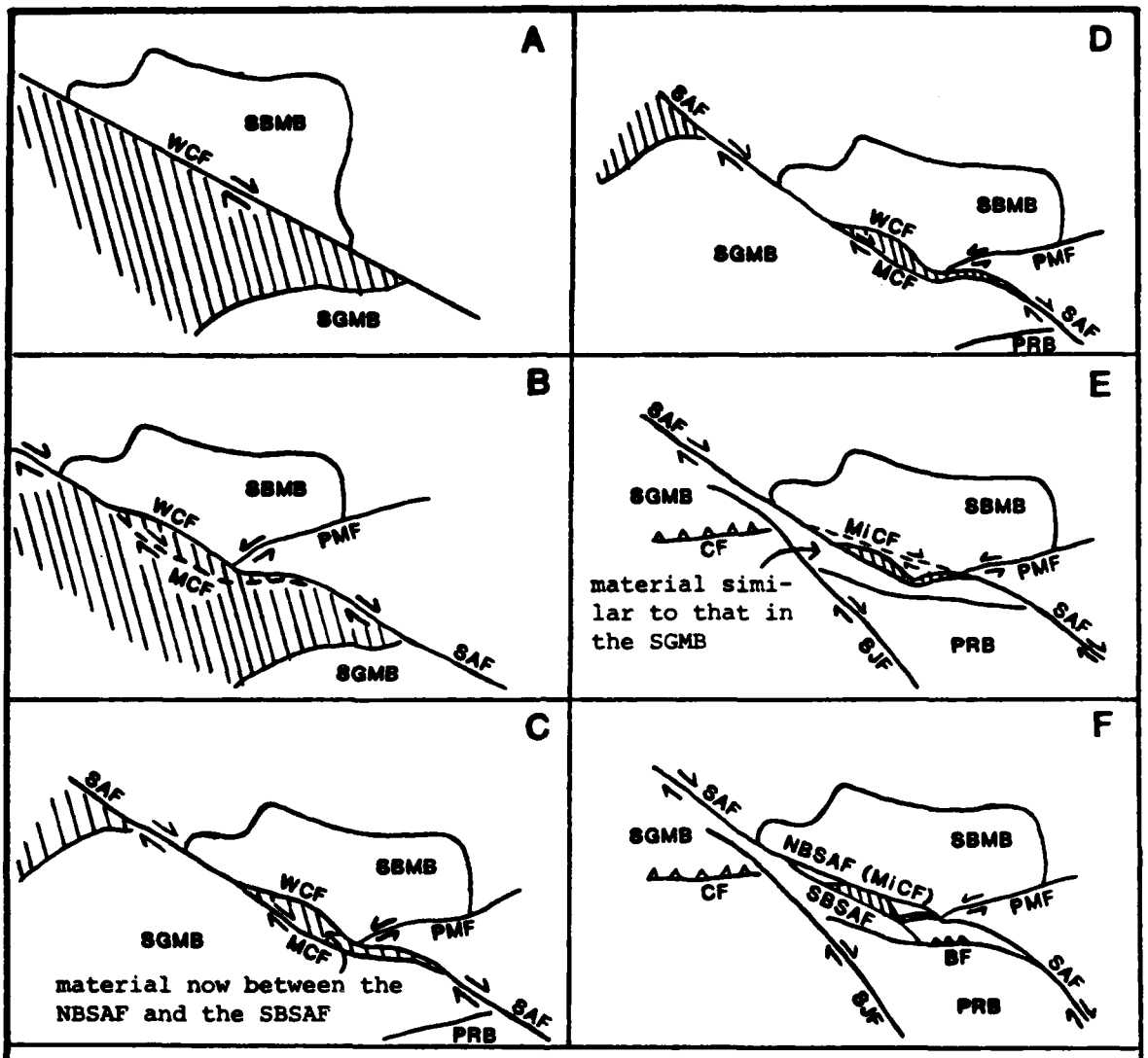
PHASE II - GENFRAL DESIGN MEMORANDUM

SEVEN OAKS DAM

Southern California Tectonics

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

FIGURE A-2



See para. 2-05 for explanation. Abbreviations: BF = Banning Fault; CF = Cucamonga Fault; MCF = Mission Creek Fault; MiCF = Mill Creek Fault; NBSAF = North Branch, San Andreas Fault; PMF = Pinto Mountain Fault; PRB = Peninsular Ranges Block; SAF = San Andreas Fault; SBMB = San Bernardino Mountains Block; SBSAF = South Branch, San Andreas Fault; SGMB = San Gabriel Mountains Block; SJF = San Jacinto Fault; WCF = Wilson Creek Fault. (After Matti and others, 1985.)

NOTE: Barbs indicate thrusting with the barbs on the upthrown side.

SANTA ANA RIVER BASIN, CALIFORNIA
SANTA ANA RIVER MAIN STEM

PHASE II - GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

San Andreas Fault Evolution

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

FIGURE A-3

V. FIELD INVESTIGATIONS

Scope of Investigations

5-01 The approved plan of study to prepare a final design for Seven Oaks Dam included more extensive exploration, testing, design, and analysis than was possible within the time and cost constraints of this GDM. It was agreed, therefore, that Phase II GDM investigations and design activities would attempt to reduce the uncertainty about the site conditions and construction materials that would have significant impacts on the cost of the dam. The remaining investigations and final design activities would be accomplished and reported in the form of separate Feature Design Memorandums (FDM's) for the embankment and spillway, and the outlet works. Therefore, in order to establish design values and prepare a construction cost estimate for the Seven Oaks Dam GDM, the following items were considered to be of primary importance:

a. Depth and configuration of the foundation bedrock surface in the canyon bottom at the damsite and outlet works portals,

b. Characteristics of the foundation bedrock,

c. Location and significance of faults and shears,

d. Location of significant landslides,

e. Static and dynamic properties of the streambed alluvium,

f. Sources and properties of construction materials, and

g. Excavatability and quality of the bedrock in the spillway and Government Canyon drainage channel.

5-02 A field and laboratory investigation program was initiated in February 1984 to specifically address these items, and the results of the initial investigations were reported in the Flood Storage Alternatives Study Supplement to the Upper Santa Ana River Phase I GDM

in December 1985. Additional investigations have been conducted since 1985 to reduce areas of uncertainty about subsurface conditions that could significantly affect the cost estimate, and to obtain suitable samples for laboratory testing. A summary of the geotechnical investigation program and procedures used is discussed in the following paragraphs.

Investigation Procedures

GEOLOGIC MAPPING, LITERATURE RESEARCH AND AERIAL PHOTO INTERPRETATION

5-03 Reconnaissance-level mapping in Santa Ana River Canyon and Warm Springs Canyon, as well as in surrounding areas accessible by vehicle, was conducted early in the feasibility investigation phase. Published geologic information relating to the damsite and vicinity was researched, and geologists knowledgeable in the area were contacted, including John Matti and Ray Weldon of the U.S. Geological Survey and Dr. Pete Sadler of the University of California at Riverside. Black-and-white and color stereoscopic aerial photographs of the general area including the reservoir at scales of 1:7,200 to 1:18,000 were reviewed. Additional literature research and aerial-photo interpretation were conducted by Schaefer Dixon Associates (1985) and Photographic Interpretation Corporation under contract with the Corps. The general damsite and reservoir area were mapped at a scale of 1-inch = 400-feet. The damsite was mapped on 1-inch = 100-feet scale field maps and herein presented at a scale of 1-inch = 200-feet. Throughout the geologic mapping program the identification of potential geologic hazards relating to landslides and faults was given special priority.

SPECIAL FAULT STUDIES - SOIL CHRONOLOGY

5-04 In order to better understand the recent tectonic history at the damsite, a special fault study was initiated in 1987 and will continue into the FDM study phase. The purpose of the study is to determine the recency of activity of the North Branch of the San Andreas fault as it passes through the reservoir area and better understand the tectonic significance of this structure. To this end, Ray Weldon and Les McFadden, through the U.S. Geological Survey, are attempting to determine the age of alluvial terraces which overlie the fault. They are using soil chronology techniques and will attempt to relate data from Upper Santa Ana River Canyon to other drainages in the San Bernardino Mountains where a history of aggradation and degradation has already been established.

TRENCHING

5-05 Exploratory trenches were excavated for different purposes at various locations within the general project site. The following paragraphs describe the different types of trenching conducted and the reasons for their excavation.

Fault Trenching

5-06 A D-8 Caterpillar dozer was used to excavate several trenches on Government Canyon Ridge in an attempt to uncover faulting related to the North Branch of the San Andreas. The purpose of the excavations was to inspect the nature of the fault contacts in the bedrock and also to determine whether overlying terrace deposits and colluvium had been displaced by fault movements. The trenching was made difficult because the overlying terrace deposits are composed of poorly consolidated clastic material with boulders up to 5 feet in diameter and are relatively thick, requiring excavations up to 30 feet deep. The trench exposures were inspected by Ray Weldon of the U.S. Geological Survey and a sketch log was made. Several soil pits approximately 10 feet deep were also excavated with the D-8 dozer into the terrace surface to aid in sample collection for the soil chronology studies conducted by Les McFadden. *See paragraphs* 5-58 to 5-61 of this section for a further discussion of fault investigations.

Foundation Bedrock

5-07 Based upon the shallow depth to rock in exploratory borings in the streambed at the toe of the left abutment and nearby bedrock outcrops, it was believed that the foundation bedrock surface could be exposed in a relatively shallow trench. A D-8 dozer was used to make a 160-foot long excavation in the coarse, streambed alluvium near hole C86-32. The location of this hole is shown on plate A-9. The trench was only excavated to a maximum depth of about 8 feet before groundwater was encountered. During excavation, only about 50 ft^2 of bedrock was partially exposed. The depression was backfilled and no further excavation to expose the foundation bedrock surface was attempted.

Foundation Alluvium

5-08 Test trenches in the foundation alluvium were excavated using a D-8 dozer because of the large percentage and size of rocks present, and the need for easy access to the trench bottom for large-scale density testing. The relatively wide and gently sloped ramps produced by dozer excavation allowed room for equipment and testing personnel to evaluate in situ foundation conditions and obtain samples for laboratory testing.

Pervious Borrow Area

5-09 A Cat 235 tracked backhoe with a 4-foot wide bucket was used to excavate test pits in the pervious borrow area in order to excavate deeper than is practical with a dozer and to obtain large samples from the wall of the pit. Because there are endangered plant species in the borrow area, the locations of which had not yet been determined, it was also desireable to limit the area disturbed by the testing. The backhoe was able to dig nearly vertical pits within areas that had been surveyed for the presence of the endangered plants, and thus disturb far less area than with conventional trenching methods. Since the deposit appeared to be uniformly distributed, the sample was taken from the full

depth of each pit by scraping one wall with the bucket and cleaning out all the material which fell into the bottom of the pit. The sample was placed upon a sheet of plastic next to the test pit by the backhoe for subsequent testing.

Impervious Borrow Area

5-10 A Case 580D backhoe was used to excavate test trenches in the impervious borrow area. The trenches were 18 inches wide and stepped at one end so that inspectors could walk into the trench and log the sides visually, obtain samples of representative materials, and conduct sand cone density tests at various depths.

DRILLING

5-11 During the Phase I and Phase II studies for Seven Oaks Dam, several types of exploratory borings were excavated. These included rotary-wash, wireline and conventional coring, bucket auger and Becker hammer.

Rotary-Wash Drilling

5-12 Rotary-wash holes, designated R84- or R86- depending on which year they were excavated, are defined in this appendix as conventionally drilled holes in which coring was not conducted. Generally, rotary-wash holes were drilled to determine the depth to bedrock and/or to install PVC pipe for use either as an observation well or a source or receiver hole for cross-hole geophysical surveys. In two cases, R86-08 and R86-40, the holes were so designated because they had to be abandoned before coring could begin. All rotary-wash holes were logged by inspecting the drill cuttings and monitoring the drilling action. Boulders and intervals of sand in excess of 1-foot thick were noted and logged. Determination of bedrock was based upon uniformity of the drill cuttings and smooth drilling action. In most cases, the top of bedrock could be determined within 0.5 foot.

Observation Wells

5-13 Observation wells were installed in rotary-wash holes R84-01, R84-02, R84-03 and R86-40. Each hole was drilled using a 5-1/8 or 5-3/8-inch diameter carbide button bit and Revert drilling fluid. Upon completion of drilling, the Revert was thinned before lowering the 3-inch diameter PVC pipe into the hole. The bottom of the pipe was capped and the lower section of the pipe was slotted using a hacksaw. The annulus was filled with sand or sand-size drill cuttings. At the surface, a 4-inch I.D. metal casing was placed over the PVC and cemented in-place. Approximately 2 feet was left protruding above the ground surface. The top was secured with a threaded metal cap. (A similar procedure was used to install an observation well in core hole C87-35.)

Geophysical Crossholes

5-14 Rotary-wash holes to be used for geophysical crosshole surveys were drilled with a Mobile B40L drill rig using a 5-5/8-inch diameter tri-cone carbide button bit. Three holes spaced approximately 10-feet apart comprise each of three crosshole sets. These holes are designated R86-01A-C, R86-02A-C and R86-03A-C. Each of the nine holes drilled for this application were drilled approximately 4 feet into bedrock before the 4-inch I.D. non-perforated PVC was grouted in-place. The grout was pumped through a 1-inch pipe and check-valve in the bottom of the PVC so that all the drilling fluid was displaced and grouting of the annulus was complete.

Core Drilling

5-15 Core drilling was the primary subsurface exploration technique used at the Seven Oaks damsite. Both conventional and wireline drilling methods were utilized. Two separate episodes of core drilling were conducted. In 1984, nine holes, C84-01 to C84-09, were conventionally cored using a CME 55 drill rig and an NQ3-size core barrel, which produces 1.775-inch core. In 1986-87, 36 additional holes, C86-01 to C86-39, were drilled by the wireline method using Failing drill rigs and either HQ or HQ3 core barrels, which produce 2.5 and 2.406-inch diameter core, respectively. The overburden in all holes was drilled by conventional rotary-wash methods using 5-1/8 to 6-3/4-inch diameter carbide button bits and either Revert or standard drilling mud. The overburden was logged by inspecting the drill cuttings and monitoring the drilling action and penetration rate. Boulders and intervals of sand in excess of 1-foot were noted and logged. Determination of bedrock was based upon uniformity of the drill cuttings and smooth drilling action. In most cases, the top of bedrock could be determined within 0.5 foot and occasionally within 0.1 foot. Generally, 2 to 4 feet of the bedrock was rotary drilled in order to be certain that the top of bedrock had been encountered before setting surface casing and beginning to core.

Core Logging

5-16 Each core run, which varied in length from 0.5 to 10.3 feet, was logged in detail in the field. The core recovery percentage and Rock Quality Designation (RQD) were calculated, the rock and discontinuities described in detail, total drilling time and longest intact core piece were recorded, and the total number of discontinuities were counted. For the discontinuity count, not only were actual breaks in the core included but also incipient weakness planes which it was felt would break during normal excavation, hauling and compaction procedures. For zones of core rubble or intensely fractured core where an accurate count could not be made, an arbitrary 10 discontinuities per foot was assumed. From the discontinuity count, a discontinuity frequency was calculated by dividing the count by the feet of recovered core. By definition, a discontinuity frequency of 10 is the maximum possible.

5-17 **DISCONTINUITY ANALYSIS.** In addition to simply counting actual or incipient breaks in the core, almost all joints, fractures and shears were described individually on the basis of their planarity, roughness, aperture and filling material. The inclination of each discontinuity as measured perpendicular to the axis of the core was also measured to the nearest 5 degrees and recorded. For each core hole, this data was sorted on the basis of inclination and a frequency relationship developed. Because more steeply inclined discontinuities are encountered less frequently in vertically drilled holes, the actual count was manipulated in the following manner. For discontinuities between 30 and 70 degrees, the actual number of discontinuities counted for a particular inclination was divided by the cosine of that inclination to yield a corrected frequency. Counts for discontinuities inclined 25 degrees or less were not corrected and counts for discontinuities inclined greater than 70 degrees were all divided by 0.33 (roughly the cosine of 70 degrees) to yield a corrected count. Furthermore, the discontinuity descriptions were used to calculate a percentage of discontinuities along which movement was indicated. Movement was defined by a slickensided surface or clay gouge filling material. This information was analyzed and included in the descriptions on the drilling logs shown on plates A-11 to A-29.

5-18 **ROCK QUALITY DESIGNATION.** The RQD was calculated for each core run by dividing the total length of intact core pieces at least 4-inches in length by the length of the core run. Although USACE practice is to apply RQD to NX-size core only, the RQD was calculated for NQ3, HQ and HQ3-size core. It might be expected that smaller diameter core (NQ3) would yield slightly lower RQD values and larger diameter core slightly higher values than for NX-size core, but based upon the results of the core drilling at the Seven Oaks site, the results were indistinguishable from each other.

5-19 **WEATHERING INDEX.** The relative degree of rock weathering was described in the field using the classification criteria shown in table A-3.

Table A-3. Classification of Degree of Weathering of Rocks (after Bieniawski 1979).

Unweathered	No visible signs of weathering; rock fresh; crystals bright.
Slightly weathered	Discontinuities are stained or discolored and may contain a thin filling of altered material. The discoloration may extend into the rock from the discontinuity surfaces to a distance of up to 20% of the discontinuity spacing.

Table A-3. (Continued)

Moderately weathered	Slight discoloration extends from disweathered continuity planes for greater than 20% of the discontinuity spacing. Discontinuities may contain filling of altered material. Partial opening of grain boundries may be observed.
Highly weathered	Discoloration extends throughout the rock and the rock material is friable. The original texture of the rock generally has been preserved, but separation of the grains or crystals has occurred.
Completely weathered	The rock is totally discolored, decomposed and friable. The external appearance of the rock sample is that of soil. Internally, the rock structure is partially preserved but grains and crystals have completely separated.

Point Load Testing

5-20 The point load test is conducted by applying compressive point loads diametrically to a specimen and measuring the load at failure. A point load strength index is derived from the failure load and the distance between the loading platens (diameter of the core). The specimen actually breaks in tension, but a relationship between the point load strength index, compressive strength and overall rock hardness appears to exist. A Rocktest Model PIL-5 Point Load Tester was used to conduct 951 individual tests on NQ3, HQ and HQ3-size core from the Seven Oaks damsite. To properly conduct the diametral test, the core piece should be at least twice as long as its diameter, therefore most pieces tested were at least 4 inches in length and could arguably represent the better quality rock. In most holes, every testable core piece was tested and the load required to break the specimen was recorded. For a test to be considered valid, the failure must occur across mineral grain boundaries and extend through both platen contact points. Therefore, failures partially or totally along shears, joints, foliation or other zones of weakness are considered invalid and are designated by an asterisk on the drill logs. Out of 951 tests attempted, 210 were determined to be invalid. The point load strength index values shown on the drill logs, plates A-11 to A-29, have been normalized to represent 50mm core.

Pressure Testing

5-21 Pressure tests were conducted in order to determine the rockmass permeability in 25 of the 45 core holes drilled at the Seven Oaks damsite. As a general rule, holes drilled into foundation rock under

the central portion of the proposed dam were pressure tested whereas holes drilled in rock to be excavated, such as the spillway or Government Canyon Ridge, were not routinely tested. Both wireline and conventionally cored holes were tested in basically the same manner. During the 1984 drilling, 10 to 20 foot intervals were tested. However, based upon the data from these holes which showed the rockmasses at the site to be relatively tight, the testing interval for the 1986 drilling was widened to anywhere from 15 to 50 feet. At each test interval, several pressures were normally used with the maximum pressure in lb/in^2 equal to the top of the test interval in feet. The water take in ft^3/min was monitored and recorded until the rate of take stabilized, usually 2 to 5 minutes. The following formula was used to calculate the permeability, K in ft/day, for each pressure used in each interval:

$$K = Cp \times Q / Ht$$

where:

Cp = conversion factor relating the length of the test interval to the size of the borehole

Q = rate of flow in ft^3/min

Ht = total pressure head in ft

A representative K value for each interval was subjectively selected after a review of data for the individual tests in that interval and is presented on the drilling logs, plates A-11 through A-29.

Bucket Auger Drilling

5-22 A power bucket auger with a 24-inch diameter bucket was used to drill test holes in the left abutment colluvium and the impervious borrow area. Changes in material type were observed and logged from the bucket contents and disturbed samples of representative materials were obtained for laboratory testing. In order to preserve the integrity of the samples, drilling fluid was not used to keep holes from caving. Standard penetration tests were not conducted because of the relatively large amounts of gravel and cobbles that were encountered.

Becker Hammer

5-23 A Becker reverse circulation drill was used in 1984 to drill nine holes to bedrock in the foundation alluvium. Several holes had to be abandoned and redrilled at adjacent locations due to the inability of the drill to penetrate large boulders. The holes were visually logged by noting the penetration rate and examining the particles removed from the hole. Eight 7-inch diameter holes were drilled, cased with 3-inch diameter PVC pipe, the annular space backfilled with sand, and used for crosshole surveys. One of the holes was reamed to 12 inches, cased with 10-inch diameter perforated PVC pipe and used in a pumping test to determine aquifer characteristics.

GEOPHYSICAL SURVEYS

5-24 Geophysical surveys at the Seven Oaks damsite consisted of magnetometer, downhole, crosshole and refractive seismic surveys.

Magnetometer

5-25 A Geometrics model G-856X Proton Precession Magnetometer was used to conduct twelve individual traverses at the Seven Oaks site. It was hoped that fault zones and formational contacts (especially between the sedimentary Mill Creek formation and the basement rocks) could be recognized by anomolies in the recorded magnetic field. With this aim in mind, traverse lines were laid out across known fault zones (Fault "A" and the North Branch of the San Andreas) and across formational contacts between the diorite and gneiss at the damsite. Field procedure was to locate the end points of the traverse on a site map and tape-off station locations every 10 to 20 feet at which readings were taken. The magnetic data was recorded to the memory of the G-856X and subsequently downloaded to a microcomputer for tabulation and plotting. An analysis of the data did not indicate any correlation between magnetic fluctuations and geologic structure and the data is not herein presented.

Refractive Seismic

5-26 Between 1984 and 1987, refractive seismic surveys were conducted in the vicinity of the Seven Oaks damsite. The surveys were primarily conducted by Los Angeles District personnel using 12-channel signal enhancement seismographs. Several preliminary surveys with 50-foot geophone spacings were conducted by Leighton and Associates. Initially, the primary purpose of the surveys was to determine the thickness of the alluvium in the canyon bottom. Subsequently, this geophysical technique was used to estimate the amount of colluvial cover on the abutment slopes and at the outlet works portals and also to determine the P-wave velocity of the rock in the spillway and Government Canyon Ridge.

5-27 Except for the contracted geophysical surveys, geophone spacings were either 15 or 30 feet with beyond-the-end and mid-line shots commonly conducted. Because of the high attenuation of the energy source in the dry surficial alluvium and colluvium, 1/3 pound to 1-pound sticks of two-component high velocity explosives were used. Record quality was generally excellent. Standard crossover-point interpretive methods were used to calculate velocity interface depths. Interpretation of surveys conducted in the canyon bottom were made difficult by the relatively high P-wave velocity of the coarse, bouldery, saturated alluvium and the relatively low velocity of the foundation bedrock. The apparently irregular subsurface bedrock relief (as determined by exploratory borings) also made interpretation of the depth to bedrock difficult both in the canyon bottom and on the abutments.

5-28 Several seismic surveys were conducted in Santa Ana River Canyon downstream from the Seven Oaks damsite during a reconnaissance exploration at an alternative damsite. The results of those surveys, which were inconclusive, are not herein presented.

Downhole Seismic

5-29 Selected Becker Hammer holes drilled by Woodward Clyde Consultants for the Corps were used to conduct both downhole and crosshole geophysical surveys in 1984, see attachment A-3. These surveys were used to determine the P and S-wave velocities in the streambed alluvium. In 1986, selected core holes drilled in the spillway and Government Canyon Ridge were used to conduct downhole geophysical surveys in order to determine the P-wave velocity of the rock to be excavated from both areas. Ten or twenty-foot long sections of 4-inch surface casing was left in the holes to be tested; otherwise the holes were uncased. A triaxial downhole geophone was pneumatically pressed against the side of the borehole at either 5 or 10-foot depth intervals. Vertical blows with a 20-pound sledgehammer at the surface were used to generate P-waves. Attempts to generate and record polarized shear waves by striking the ends of a shallow trench were unsuccessful. The P-wave first arrivals were picked to within 0.1 millisecond and recorded. The maximum depth which could be tested was approximately 140 feet due to attenuation of the surface energy source.

Crosshole Seismic

5-30 Crosshole seismic testing was conducted by Woodward-Clyde Consultants in three sets of three holes in the streambed foundation alluvium that were advanced with a Becker Hammer drill rig. Crosshole shear wave velocities between the holes were determined at 5-foot intervals in each hole by placing a reversible shear-wave hammer in the end boring of each set and measuring the travel time required for shear waves to arrive at the other boreholes. The hammer and geophones were clamped to the borehole walls pneumatically and an inclinometer was used to measure deviations from vertical to obtain true distances between geophone locations in the boreholes.

Summary of Explorations

STREAMBED FOUNDATION

5-31 Explorations in the streambed were conducted to determine the material properties of the streambed alluvium, material properties of the streambed foundation bedrock and the configuration of the covered bedrock surface. The explorations included, trenching, drilling and geophysical surveys. A description of those explorations is presented in the following paragraphs.

Drilling

5-32 Drilling in the streambed included core borings, rotary-wash borings, and Becker Hammer holes. Some of the rotary-wash borings and one of the core holes were cased and used as observation wells or source/receiver holes for geophysical surveys. The Becker Hammer holes were cased with perforated PVC pipe and used for downhole/crosshole geophysical surveys and in an aquifer test. See plate A-9 for the location of the holes. Pertinent hole data are listed in table A-4. Holes used as permanent observation wells are designated with an asterisk (*) before the hole number.

Table A-4. Streambed Drilling.

Hole No.	Date Completed	Hole Elevation (ft)	Depth Drilled (ft)	Length Cored (ft)
C84-01	08/15/84	2053.8	104.8	15.0
C84-02	09/07/84	2052.7	190.1	100.3
C84-03	09/10/84	2070.1	167.4	72.1
C84-04	09/21/84	2064.8	102.4	36.9
C84-05	09/28/84	2072.1	177.5	83.9
C86-02	10/07/86	2073.4	100.0	79.6
C86-05	10/17/86	2109.1	108.6	48.6
C86-26	02/02/87	2051.3	302.5	218.5
C86-28	02/07/87	2054.2	122.6	49.1
C86-29	02/12/87	2054.0	81.0	51.0
C86-32	04/21/87	2073.4	51.7	41.7
C86-33	04/25/87	2079.7	151.8	51.8
C86-34	04/28/87	2064.6	81.4	50.2
*C86-35	05/05/87	2068.9	92.0	52.5
C86-36	05/01/87	2070.7	109.5	50.0
C86-37	05/06/87	2074.1	99.1	48.1
C86-38	05/08/87	2066.2	90.3	3.5
C86-39	05/08/87	2067.0	80.5	10.0
*R84-01	06/01/84	2065.0	70.0	-
*R84-02	06/07/84	2057.8	67.5	-
*R84-03	06/12/84	2054.8	96.0	-
R86-01A	10/09/86	2040.0	83.0	-
R86-01B	10/14/86	2040.0	69.0	-
R86-01C	10/20/86	2040.0	60.0	-
R86-02A	10/23/86	2062.0	74.0	-
R86-02B	10/28/86	2062.0	75.0	-
R86-02C	10/31/86	2062.0	77.0	-
R86-03A	11/14/86	2096.0	105.0	-
R86-03B	11/10/86	2096.0	104.0	-
R86-03C	11/05/86	2096.0	101.0	-

Table A-4. (Continued)

Hole No.	Date Completed	Hole Elevation (ft)	Depth Drilled (ft)	Length Cored (ft)
*R86-40	05/04/87	2102.0	49.5	-
R86-41	05/02/87	2080.7	41.0	-
B84-01	10/05/84	2069.6	65.3	-
B84-02	10/11/84	2069.2	95.3	-
B84-05	10/15/84	2069.9	95.0	-
B84-06	10/16/84	2066.7	79.0	-
B84-07	10/17/84	2067.4	85.0	-
B84-09	11/02/84	2068.0	80.0	-
B84-11	11/05/84	2072.9	16.5	-
B84-12	11/06/84	2072.5	13.0	-
B84-13	11/06/84	2072.6	19.0	-
Total Footage:			3833.3	1063.1

Geophysical Surveys

5-33 Refractive seismic, downhole and crosshole geophysical surveys were conducted in the streambed foundation area between April 1984 and March 1987. Very early in the feasibility investigation phase of the project, four refractive seismic surveys using 50-foot geophone spacings were conducted in the streambed at the damsite by Leighton and Associates, see attachment A-4. These preliminary surveys were supplemented by eleven shorter lines conducted by Los Angeles District personnel. Downhole and crosshole surveys were conducted in three sets of Becker holes (B84-01, 02, 05), (B84-06, 07, 09) and (B84-11, 12, 13) located near the dam centerline. Three additional crosshole sets of three holes each were drilled in 1986 (R86-01, 02 and 03). These sets are located at the downstream toe, centerline and upstream toe respectively. Crosshole and downhole geophysical surveys will be conducted in these holes during the Embankment FDM investigations. See plate A-10 for the locations of the refractive seismic lines and plate A-9 for the locations of the crosshole sets.

Trenching

5-34 Three trenches (TT84-1,2 and 3) were excavated at representative locations in the streambed during October 1984 to determine the gradation and in-place density of the materials in the foundation of the proposed embankment. One test trench was excavated to a depth of 22 feet with a Cat 977 loader, and 2 other test trenches were excavated to depths of 22 and 17 feet with a D-8 dozer. All three trenches were terminated approximately 1 foot below the elevation where groundwater was encountered. The walls of the test trenches were visually logged and large-scale density tests conducted at 5-foot intervals to obtain gradation and density samples.

5-35 Four more test trenches were excavated in the streambed in August, 1987 as part of the concrete aggregate exploration program. Although only one trench, TT87-2, is located within the embankment toe trace, two others, TT87-1 and 3, are within about 500 feet of the downstream and upstream toes, respectively, and can be considered representative of near surface foundation materials under the embankment. The other trench, TT87-4, is located about 1/4 mile upstream of the damsite and would be representative of materials that would be obtained from the upstream pervious borrow area. The location of the trenches is shown on plate A-97. The trenches were excavated with a D-8 dozer to depths ranging from 8 to 13 feet. A sample of representative materials from each trench was obtained for gradation and rock quality testing using a 955K tracked loader. Each sample weighed approximately 15,000 pounds and included rocks up to about 3 feet in diameter. Percentages and maximum sizes of larger rocks were visually estimated and noted on the logs.

RIGHT ABUTMENT

5-36 Because of the steep and generally inaccessible terrain, minimal exploration, consisting of five core holes, was conducted near the base of the right abutment. No geophysical surveys were conducted. After abutment excavation and construction of an access road system, several deep core holes will be drilled during Embankment FDM studies.

Drilling

5-37 Five core holes have been drilled on or at the toe of the right abutment. Two holes, C86-01 and C86-03, were actually drilled in the streambed near the base of the steepest rock slopes. The purpose of these holes was to determine the subsurface bedrock slope. C84-08 is an angle hole drilled into the diorite rockmass at the base of the abutment, downstream of centerline. Holes C84-07 and C86-23 were drilled into the gneissic rockmass upstream from centerline. See plate A-9 for the hole locations. Pertinent hole data are listed in table A-5.

Table A-5. Right Abutment Drilling.

Hole No.	Date Completed	Hole Elevation (ft)	Depth Drilled (ft)	Length Cored (ft)
C84-07	12/14/84	2090.0	201.5	171.4
C84-08	12/31/84	2057.0	84.5	79.2
C86-01	10/08/86	2050.0	73.4	48.0
C86-03	10/11/86	2058.2	82.1	52.1
C86-23	01/24/87	2159.0	312.4	302.2
Total Footage:			753.9	652.9

Trenching

5-38 No test trenches were excavated in the area of the right abutment. The access road to hole C86-23, however, created up to 20-foot high exposures of the gneissic rockmass. These exposures aided in the geologic mapping of the abutment site.

LEFT ABUTMENT

5-39 Explorations on the left abutment consisted of core drilling, bucket auger drilling, and refractive seismic surveys. One of the main purposes of the explorations conducted to date was to more accurately determine the thickness and physical properties of the overburden materials which are much more extensive than at the right abutment.

Drilling

5-40 Only two core holes, C84-06 and C86-04, and four bucket auger holes, LAH-01 to LAH-04, were drilled at the left abutment. Core hole C86-04 and the four 24-inch diameter bucket auger holes were drilled through the thick mass of colluvium under the centerline of the proposed dam. Hole C84-06 was drilled downstream from centerline at the toe of the gentle abutment slope. The auger holes were terminated when bedrock was encountered or caving prevented further drilling. The holes were logged and representative samples of disturbed materials were collected for classification and detailed laboratory testing. Exact locations of the bucket auger and core holes are plotted on plate A-9. Pertinent hole data are presented in table A-6.

Table A-6. Left Abutment Drilling.

Hole No.	Date Completed	Hole Elevation (ft)	Depth Drilled (ft)	Length Cored (ft)
C84-06	12/04/84	2060.0	192.3	158.1
C86-04	10/24/86	2215.3	400.5	284.5
LAH-01	11/25/86	2215.0	90.0	-
LAH-02	11/26/86	2280.0	47.5	-
LAH-03	11/26/86	2265.0	44.0	-
LAH-04	12/01/86	2215.0	35.0	-
Total Footage:			809.3	442.6

Geophysical Surveys

5-41 Two refractive seismic surveys, 85-4 and 86-09, were conducted on or at the toe of the left abutment. See plate A-10 for the location of the survey lines.

Trenching

5-42 No test trenches were excavated on the left abutment. The access road to the spillway site for the drill rigs, however, created up to 20-foot high exposures of the diorite and gneissic rock. These exposures aided in the geologic mapping of the abutment.

SPILLWAY

5-43 Explorations at the spillway site consisted of core drilling and geophysical surveys. The purpose of the explorations was to determine the excavatability of the rockmass and estimate the size and quality of the rock produced for use in the dam embankment. Information from the explorations along with laboratory test data was used to make a comparison between the Government Canyon Ridge and spillway site rockmasses.

Drilling

5-44 Eight HQ3 wireline core holes were drilled in the area of the proposed spillway excavation. Two holes, C86-15 and C86-17, were drilled only a few feet from each other because equipment malfunction caused very poor core recovery in hole C86-15. See plate A-9 for the hole locations. Each hole was drilled to the proposed grade cuts for the spillway excavation. Pertinent hole data are listed in table A-7 and hole locations are shown on plate A-9.

Table A-7. Spillway Drilling.

Hole No.	Date Completed	Hole Elevation (ft)	Depth Drilled (ft)	Length Cored (ft)
C86-15	11/28/86	2704.0	151.1	141.8
C86-16	12/01/86	2969.7	295.8	285.8
C86-17	12/01/86	2704.0	127.1	117.4
C86-18	12/08/87	2911.6	329.6	319.6
C86-19	12/09/86	2965.2	245.2	235.6
C86-20	12/12/86	2780.0	210.1	200.6
C86-21	12/11/86	2716.0	128.0	117.6
C86-22	12/15/86	2686.0	123.5	113.0
Total Footage:			1610.6	1531.4

Geophysical Surveys

5-45 Two refractive seismic surveys, 86-11 and 87-01, were conducted on the spillway saddle ridgecrest and upstream slope, respectively. See plate A-10 for the location of the survey lines. Downhole surveys were conducted to depths up to 140 feet in holes C86-19, C86-20 and C86-21.

Trenching

5-46 No test trenches were excavated in the area of the proposed spillway excavation, however, the access road for the drill rigs, created exposures of the gneissic rock up to 15 feet high. These exposures aided in the geologic mapping of the spillway site.

GOVERNMENT CANYON RIDGE

5-47 Explorations on Government Canyon Ridge consisted of core drilling and geophysical surveys in the area of the proposed drainage channel excavation. The purpose of the explorations was to determine the excavatability of the rockmass and estimate the size and quality of the rock produced for use in the dam embankment. Information from the explorations was also used to plan the test excavation and test fill to be conducted during Embankment FDM studies. Information from the explorations along with laboratory test data was used to make a comparison between the Government Canyon Ridge and spillway site rockmasses.

Drilling

5-48 Four HQ wireline core holes were drilled on Government Canyon Ridge in the area of the proposed drainage channel excavation. Three holes were drilled on the ridge crest and one drilled on the northwest side of the ridge. See plate A-9 for the hole locations. Access to the drill sites was very difficult for the truck mounted Failing drill rig. Each hole was drilled to the proposed grade cuts for the drainage channel excavation. Pertinent hole data are listed in table A-8.

Table A-8. Government Canyon Ridge Drilling.

Hole No.	Date Completed	Hole Elevation (ft)	Depth Drilled (ft)	Length Cored (ft)
C86-09	11/01/86	2434.0	300.2	290.5
C86-11	11/13/86	2391.0	270.4	260.4
C86-14	11/21/86	2337.0	221.5	211.5
C86-27	01/29/87	2259.0	119.7	110.1
Total Footage:			911.8	872.5

Geophysical Surveys

5-49 One refractive seismic survey, 86-10, was conducted on the ridgecrest in the vicinity of drill hole C86-11. Downhole surveys were conducted to depths of 180 and 120 feet in holes C86-14 and C86-27, respectively. See plate A-10 for the location of refractive seismic survey 86-10.

Trenching

5-50 No test trenches were excavated on Government Canyon Ridge in the area of the proposed excavation. The access road for the drill rigs, however, created up to 15-foot high exposures of the gneissic rock. These exposures aided in the geologic mapping of the Government Canyon Ridge site.

OUTLET WORKS

5-51 Explorations for the outlet works were confined to the inlet and outlet portal and energy dissipator sites. A combination of drill holes and refractive seismic surveys were used to determine the overburden thicknesses and the quality of rock for use as a foundation for outlet structures.

Drilling

5-52 Drilling for the outlet works consisted of two core holes and two rotary holes at the inlet portal area, four core holes and one bucket auger hole at the outlet portal and one rotary and one core hole in the energy dissipator. Hole C86-24 at the outlet portal was an angle hole inclined 60 degrees N50E. All of the rotary-wash holes were originally planned as core holes, however, the depth to bedrock in holes R86-06 and R86-31 was deeper than anticipated and hole R86-08, in the energy dissipator area, was abandoned at 68.5 feet when the roller bit broke off in the hole. The hole locations are plotted on plate A-9. Data from pertinent holes are presented in tables A-9, A-10, and A-11.

Table A-9. Outlet Works - Inlet Portal Drilling.

Hole No.	Date Completed	Hole Elevation (ft)	Depth Drilled (ft)	Length Cored (ft)
R86-06	10/22/86	2136.7	72.8	-
C86-07	10/25/86	2136.6	90.8	49.3
C86-30	04/16/87	2121.9	120.4	43.9
R86-31	04/18/87	2137.5	101.0	-
Total Footage:			385.0	93.2

Table A-10. Outlet Works - Outlet Portal Drilling.

Hole No.	Date Completed	Hole Elevation (ft)	Depth Drilled (ft)	Length Cored (ft)
C86-12	11/13/86	2135.0	102.0	27.7
C86-13	11/15/86	2135.0	76.1	15.5
C86-24	01/22/87	2135.0	119.4	85.9
C86-25	01/24/86	2135.0	48.4	6.8
LAH-05	12/01/86	2130.0	19.0	-
Total Footage:			364.9	135.9

Table A-11. Outlet Works- Energy Dissipator Drilling.

Hole No.	Date Completed	Hole Elevation (ft)	Depth Drilled (ft)	Length Cored (ft)
R86-08	10/28/86	2021.0	68.5	-
C86-10	11/04/86	2021.0	126.8	6.8
Total Footage:			195.3	6.8

Geophysical Surveys

5-53 Two refractive seismic surveys, 87-03 and 87-05, were conducted in the area of the inlet portal in an attempt to determine the configuration of the bedrock under the proposed intake structure. Similarly, two surveys, 86-03 and 86-08, were conducted on the colluvium covered slopes in the area of the outlet portal in order to determine the depth to bedrock. One long refractive survey, 84-1/2, was conducted for the same reason in the area of the proposed energy dissipator. See plate A-10 for the location of the geophysical survey lines.

Trenching

5-54 No test trenches were excavated in conjunction with the outlet works exploration program.

DOWNSTREAM PERVIOUS BORROW AREA

5-55 Five test pits were excavated in the Santa Ana River Wash streambed alluvium about 1 mile downstream from the canyon mouth in October 1986. The locations of the test pits are shown on plate A-55.

The pits were excavated to depths ranging from 18 to 24 feet with a Cat 235 tracked backhoe with a 48-inch wide bucket. After excavation, the pit walls were logged and photographed, and a sample of approximately 40,000 lbs was taken from each pit.

IMPERVIOUS BORROW AREA

5-56 The Mill Creek core borrow area was selected from among five sites investigated for the Flood Storage Alternative Study as the most suitable source of impervious core material for the proposed Seven Oaks Dam. The factors considered in choosing a borrow source were:

a. Physical properties of the material,

b. Available transportation system near the borrow site,

c. Distance from the damsite,

d. Cultural features at the borrow site, and

e. Site drainage.

5-57 The borrow site is located on the east side of the Santa Ana River canyon mouth, between Mill Creek and the mountain slopes, about 3 miles from the damsite. The initial investigation in June 1985 consisted of drilling 14 test holes with a 24-inch diameter bucket auger to depths ranging from 8 to 47 feet, and excavating 14 test trenches with a backhoe to a maximum depth of 16 feet. In December 1986, an additional 38 test holes, between 4 and 65 feet deep, and 34 test trenches were excavated with the same equipment. The location of the test holes and test trenches are shown on plate A-60. The holes and trenches were logged and disturbed samples of representative materials were collected for classification and detailed laboratory tests. To allow a comprehensive testing program to be conducted on similar soil types, 21 large samples of potential core materials, weighing approximately 700 lbs each, were collected in 55-gallon drums. In order to evaluate the shrinkage factor to be expected during construction, 12 more trenches were excavated in the preferred portion of the borrow area in December 1987. The trenches were logged and sampled and a total of 34 sand cone density tests were conducted at various depths in the impervious soils that are expected to be used in the embankment. Large bag samples of material corresponding to the layer in which the density test was conducted were collected in order to determine maximum density and in situ percent compaction of the materials encountered.

FAULT INVESTIGATIONS

5-58 Trenching across the North Branch of the San Andreas fault was undertaken in an attempt to better understand the components of the fault zone within the project vicinity and to establish recency of faulting. As depicted on plate A-48, the first trench site (T87-1) was located along a gully on the middle terrace on Government Canyon

Ridge. The trench was 350 feet in length and up to 25 feet deep. The thickness of colluvium and terrace deposits were much greater than anticipated and the fault contact between quartz monzonite and Tertiary sediments was not uncovered. The work was stopped and the trench backfilled when it became apparent that the bedrock would continue to dip away from the surface and the wall of the trench was becoming unsafe. The trench was subsequently re-excavated to a depth of about 8 feet in order for Ray Weldon of the U.S. Geological Survey (USGS) to inspect and map in detail the finer grained colluvium in the area overlying the projection of the fault.

5-59 Trench T87-2 was located at the edge of a hanging valley that rises 170 feet steeply above the Santa Ana River canyon. The cut is about 120 feet long, up to 30 feet wide and as much as 40 feet high. There was about an equal volume of finer grained colluvium and terrace deposits moved, but the terrace deposits are comprised of boulders as large as 10 feet in diameter that required a large amount of time and effort to dislodge and roll aside. It is believed that further excavation would have exposed the Mill Creek formation in fault contact with the gneiss. Unfortunately, continued excavation in T87-2 was logistically impossible because of the steep slopes and large, loose boulders.

5-60 Two more trenches were excavated in an attempt to expose the contact between the Mill Creek formation and the gneiss on the south side of the North Branch fault zone. Trench T87-6 was located along the top of the northeast end of Government Canyon Ridge. Fault contact between the Mill Creek formation (Tmc) and quartz monzonite (qm) was exposed at the northeast end of the trench, but continued excavation towards the southwest required deeper and deeper trenching through colluvium and terrace deposits. Eventually, the trench was 20 feet deep with no sign of the gneissic rock. The work was stopped and the trench backfilled when the trench walls became too steep to safely continue. Trench T87-7 was located in Government Canyon. Initially the trench was excavated with a dozer to a depth of approximately 17 feet in what turned out to be a landslide mass. Subsequently, a backhoe was used to dig a 6-foot deep trench in the floor of the excavation in order to expose the base of the slide and underlying structure.

5-61 Seven soil pits were excavated in colluvium to establish a soil profile chronology that in turn might help in determining the recency of faulting on the North Branch of the San Andreas fault (see para. 5-04). Most of the pits were approximately 10 feet deep and excavated with a D-8 dozer. One site was inaccessible to machinery, requiring the pits to be excavated by hand tools to a maximum depth of 8 feet. The pits were spread out for 2,000 feet along the ridge and benches on the north side of the Santa Ana River canyon, on each side of the fault zone, and were located on each of the three terraces.

FEATURE DESIGN INVESTIGATIONS

5-62 Additional field investigations are required in order to complete the approved plan of study for Seven Oaks Dam. The data from the investigations will be incorporated in the Feature Design Memorandums for the outlet works and the embankment and spillway. The scope of remaining investigations for each project feature is discussed in the following paragraphs.

Streambed Foundation

5-63 The majority of the embankment foundation investigation will be included in the proposed test fill contract activities because of the cost and the heavy equipment necessary for both operations. It will include the excavation of three test pits to depths of about 40 feet, with large-scale density tests conducted at various depths, geophysical testing, pumping tests, and comparison of in situ densities with laboratory maximum and test fill densities determined on the excavated materials. A complete discussion of the proposed test excavation and test fill program begins in paragraph 13-03 of this appendix.

5-64 Additional rotary and core drilling will be conducted in the streambed during the FDM study phase in order to refine the bedrock contours, the diorite/gneiss contact, and the streambed foundation rockmass permeability. Special emphasis will be placed on establishing the rockmass permeability at depth between 50 and 300 feet in order to more precisely design the foundation grouting program and refine the cost estimate.

Right Abutment

5-65 Subsurface investigation of the right abutment will be conducted after the surface is stripped and an access road system constructed as described in paragraph 12-17 of this appendix. Several core holes, both vertical and angled, will be drilled into the right abutment. Rockmass permeability values from the core holes will be used in the detailed design of the foundation grout curtain. Excavation at the abutment will allow detailed mapping of the rock surface, especially in the core contact area. Information from the mapping and inspection of the rock surface will allow for a detailed foundation treatment plan and cost estimate.

Left Abutment

5-66 Excavation of the left abutment as a part of an early construction contract will allow detailed mapping of the entire abutment rock surface and development of a refined cost estimate for foundation surface treatment. In addition, several core borings on the abutment, in conjunction with exploration for the outlet works tunnel, will provide the basis for a detailed foundation grouting plan and cost estimate.

Spillway

5-67 Several core holes will be drilled in the proposed spillway in order to verify the continuity of rockmass properties as determined in the Phase II studies and presented in this appendix. Video borehole camera surveys will be attempted in at least two holes. Information on the orientation of the discontinuities in the boreholes as well as more detailed mapping of bedrock surface exposures will allow for a more definitive cut-slope design and construction cost estimate.

Government Canyon Ridge

5-68 As a part of a test fill and test excavation contract, a portion of the ridge in the area of the proposed drainage channel will be excavated, hauled, placed, and compacted. This procedure will provide information on the excavatability and durability of the rock and stability of the cut-slopes. In addition, three core holes will be drilled from the ridgecrest in the area proposed for use in the downstream transition zone in order to verify continuity of rockmass properties.

Outlet Works

5-69 For the Feature Design Memorandum, exploration of the outlet works will consist of core drilling and adits with borehole camera surveys, in situ borehole testing, residual stress testing, rock permeability testing and laboratory rock testing. The core drilling program is general and will consist of horizontal holes at the two portals and vertical and inclined holes along the tunnel alignment. The holes will initially determine general rock conditions in the areas of proposed structures for the outlet including presence of major shears and the permeability of the rock mass needed for evaluation of the hydrostatic loading on the tunnel. The horizontal holes will also provide information needed for excavation of the adits, one at each portal. The adits will extend 200 to 500 feet into the hill and will be constructed after the alluvium (colluvium) has been removed during the test fill contract described in paragraph 5-66. The adits will allow close observation of the bedrock conditions such as shears, joint orientations, weak zones, etc., which are vital to the portal and gate chamber designs and which may not be retrieved in the drill hole cores. Flat jack tests would be conducted in the adits to determine the residual stresses in the hill which could affect the tunnel support design. Subsequent to the FDM studies, it is recommended that the adits be joined so that a complete exploratory tunnel would be available for inspection which could reduce construction contingencies and cost escalation and result in better contractor bids.

Downstream Pervious Borrow Area

5-70 A test pit will be excavated to a depth of approximately 50 feet to confirm the suitability of materials to the full depth of expected borrow excavation. Inspection trenches will be excavated during the advertising period to allow prospective bidders to evaluate the nature of the borrow materials.

Impervious Borrow Area

5-71 No further field investigations are proposed for the impervious borrow area.

VI. LABORATORY TESTING PROGRAM FOR ROCK QUALITY

Purpose and Scope

6-01 A series of laboratory tests was conducted by the South Pacific Division Laboratory on core samples from the Seven Oaks site. The tests included unconfined compressive strength, specific gravity and absorption, wetting and drying, Los Angeles abrasion and petrographic analysis. The purpose of the tests was twofold. First, unconfined compressive strength tests were conducted, mainly on outlet works and foundation rock, in order to determine design values for the bedrock and to correlate results with point load strength data. Second, the remaining rock quality tests were conducted primarily on core samples from the spillway and Government Canyon Ridge in order to better estimate the suitability of these materials for use in the embankment and better predict the amount of rock breakdown during excavation, placement, and compaction.

Unconfined Compressive Strength

6-02 Initially, it was planned on conducting at least one laboratory unconfined compression test on every 50 feet of recovered core. However, because of the highly broken nature of the bedrock and numerous incipient planes of weakness, it was difficult to find core pieces suitable for testing (isotropic and at least two diameters in length). In all, 27 samples were sent to the South Pacific Division Laboratory for compressive strengh tests. Out of these samples, 19 valid tests were performed. In some instances the samples broke during shipment or preparation for testing. In five cases, test results were questionable because the samples broke along seams, veins or foliation. Where the samples were long enough, a point load test was conducted in the field on one end before sending the sample to the laboratory. It was felt that tests on these samples would give the most valid correlation between compressive strength and point load strength indices.

Rock Quality Tests

6-03 For the spillway and Government Canyon excavations, the main purpose of the field investigations, field and laboratory testing was to determine the size and quality of the excavated rock. Because a test excavation and test rock-fill was beyond the scope of the Phase II GDM explorations, an attempt was made to indirectly determine the quality of the rock by quantitative means. Previous studies as reported by Wilson and Marsal (1979) used water absorption and Los Angeles abrasion tests to evaluate the soundness of mineral grains. This in turn is related to grain breakage in a rockfill section. The rock quality testing program for the spillway and Government Canyon Ridge included specific gravity and absorption, Los Angeles abrasion, wetting and drying tests and petrographic analyses.

SAMPLING PROCEDURE

6-04 For sampling purposes it was assumed that point load test results could be used as a qualitative index of overall rock quality. Since approximately 20 pounds or 5 feet of core were required for each abrasion test, it was decided to send entire core boxes containing approximately 11 feet of core to the laboratory. After the abrasion tests, four samples for specific gravity and absorption tests, one or two samples for wetting and drying and one "representative" sample for petrographic analysis were selected from the remaining core. Drill logs of eleven core holes drilled in the spillway and Government Canyon Ridge were analyzed to select one or two boxes from each hole containing core intervals with point load tests at or below the average point load strength for that hole. Because the point load test can only be performed on the longer (and presumably better quality) core pieces, the average point load strength for a hole is actually the average for the better quality core from that hole: core lost or highly broken can not be tested. Therefore, core with below "average" strength is believed to be a more true representation of all the core from a hole.

VII. RESULTS OF INVESTIGATIONS AND TESTING

Core Drilling - General

7-01 The results of the RQD and core recovery percentage calculations for each core run are presented on the drill logs on plates A-11 through A-29 Individual point load strength index results are also depicted on the drill logs along with interval permeabilities as determined from pressure tests. RQD and point load strength values are presented graphically on plates A-30 through A-35 along with a plot of discontinuity frequency and a qualitative depiction of the degree of weathering. These four parameters were chosen to represent the overall rock quality of the cores. From the graphs, it is possible to identify zones of poorer quality rock (low RQD, high discontinuity frequency, low strength and high degree of weathering) especially relative to the average values for each individual hole. The results of the core drilling as well as other investigations and testing are presented in this section according to specific project features.

Streambed Foundation Bedrock

DRILLING SUMMARY

7-02 Logs of holes drilled in the streambed are presented on plates A-11 through A-17. A summary of drilling in the streambed foundation is presented in table A-12.

Table A-12. Streambed Drilling Summary.

Hole No.	Rocktype	Bedrock Depth (ft)	Recovery (%)	RQD (%)	Discon. Freq. (per ft)	Point Load (lb/in^2)	K (ft/day)
C84-01	Diorite	82.5	72	3	9.4	122	0.00
C84-02	Diorite	79.7	88	22	7.3	155	0.10
C84-03	Diorite	85.0	94	42	5.4	217	0.32
C84-04	Diorite	63.5	96	38	6.9	169	0.01
C84-05	Diorite	87.7	77	5	7.5	116	0.01
C86-02	Dior/gn	17.5	98	38	5.5	146	0.03
C86-05	Gneiss	56.5	97	21	3.6	-	-
C86-26	Diorite	82.5	63	9	6.9	122	0.01
C86-28	Diorite	72.2	58	10	9.6	73	0.02
C86-29	Gneiss	25.5	89	35	6.3	168	0.08
C86-32	Diorite	8.5	96	51	3.8	181	0.00
C86-33	Gneiss	94.7	94	23	3.8	207	0.02
C86-34	Diorite	30.0	100	49	3.4	237	0.10
C86-35	Gneiss	36.0	89	35	6.6	139	0.14
C86-36	Gneiss	53.7	87	18	6.5	135	0.00
C86-37	Gneiss	49.0	83	15	5.5	99	0.00
C86-38	Gneiss	84.0	63	0	9.6	-	-
C86-39	Gneiss	69.5	83	41	4.6	133	-
R84-01		49.6					
R84-02		46.5					
R84-03		78.5					
R86-01A		83.0					
R86-01B		63.0					
R86-01C		56.0					
R86-02A		70.0					
R86-02B		71.0					
R86-02C		72.5					
R86-03A		101.0					
R86-03B		100.0					
R86-03C		96.0					
R86-41		33.5					
Overall Average:		64.5	85	25	6.2	151	0.06
Diorite Average:		65.8	83	25	6.7	155	0.06
Gneiss Average:		58.6	86	25	5.8	147	0.06

GEOPHYSICAL SURVEY RESULTS

7-03 The results of the geophysical surveys conducted in the streambottom are presented in the following paragraphs.

Refractive Seismic

7-04 The time-distance graphs and subsurface interpretations for the 15 refractive seismic survey lines conducted in the streambed are presented on plates A-36 and A-39. Basically, three significant velocity zones are identifiable. Layer 1 (V1) represents dry, unconsolidated alluvium or colluvium; layer 2 (V2) represents dense, saturated, coarse clastic alluvium; and layer 3 (V3) represents foundation bedrock. The measured velocities in feet per second and generalized depths in feet (D) for the layers identified in the streambed refractive seismic surveys are shown in table A-13.

Table A-13. Streambed, Results of Refractive Seismic Surveys.

Line No.	V1 (ft/s)	D1 (ft)	V2 (ft/s)	D2 (ft)	V3 (ft/s)
L-1	1,600-2,500	10	6,500-7,500	80	12,500
L-2	2,500	20	6,000-7,000	80	12,000
L-3	1,800	15	6,000-8,500	70	11,500
L-4	1,700-1,800	15	8,000-8,500	80	10,750
84-3	800-2,000	10	8,200		
84-6/7	800	5	8,000	50	11,000
84-8/9	1,400	10	7,600		
85-1/2	800-2,500			25	9,800
85-5	1,400	5	7,200	25	11,000
86-01	1,900	15	8,240		
86-02	1,800	10	8,750	80	12,700
86-05	700-4,400	20	8,500		
86-07	1,000-2,000	20	8,100	135	16,000
87-02	1,400	10	7,900	60	12,000
87-04	1,400	5	8,000	40	11,000

Downhole Seismic

7-05 Downhole geophysical surveys to measure the P-wave velocity of the streambed foundation bedrock were not conducted.

LABORATORY TEST RESULTS

7-06 The results of laboratory tests on samples from the streambed foundation bedrock are presented in the following paragraphs.

Unconfined Compressive Strength

7-07 Unconfined compression tests were conducted on 8 samples from streambed foundation core holes. The results of those tests are presented below in table A-14. If a point load test was conducted on

one end of a core piece sent to the laboratory, that value and the ratio of compressive strength to point load index strength is also presented. Compressive strength values with an asterisk (*) are of questionable validity because of failure along veins or foliation.

Table A-14. Streambed Foundation Bedrock, Results of Compressive Strength Tests.

Hole No.	Rocktype	Depth (ft)	Unconfined Compressive Strength (lb/in^2)	Point Load Index (lb/in^2)	Ratio
C84-03	Diorite	166.8	5,960	302	19.7
C86-26	Diorite	126.4	3,463	-	-
C86-29	Gneiss	77.1	7,833	-	-
C86-32	Diorite	28.2	6,032	302	20.0
C86-32	Diorite	33.1	3,565*	170	21.0
C86-32	Diorite	42.3	4,787*	227	21.1
C86-34	Diorite	51.6	6,641	391	17.0
C86-34	Diorite	77.2	2,446*	139	17.6
Average:			5,091		19.4

Rock Quality

7-08 The gneiss sample from hole C86-29 was also tested for specific gravity and absorption with the following results:

Table A-15. Streambed Foundation Bedrock, Results of Rock Quality Tests.

Hole No.	Rocktype	Depth (ft)	Specific Gravity (bulk SSD)	Absorption (%)
C86-29	Gneiss	77.1-77.8	2.75	0.71

Petrographic Analysis

7-09 Samples from the streambed foundation bedrock were not petrographically analyzed.

FIELD TESTS AND RESULTS

7-10 The field tests conducted in the streambed alluvium at the damsite included seismic surveys, density tests and a pumping test to determine aquifer characteristics.

Refractive Seismic Surveys

7-11 The refractive seismic surveys performed by both CESPL personnel and Leighton and Associates indicate two distinct layers within the streambed alluvium corresponding to unsaturated and saturated alluvium. The surficial, dry, unconsolidated granular alluvium exhibits characteristically low P-wave velocities in the range between 500 and 1,400 ft/s. The coarse, granular alluvium below the water table yields compressional wave velocities between 6,000 and 8,200 ft/s, but mostly closer to 8,000 ft/s. For saturated alluvium, these velocities are relatively high and could be due to the concentration of cobbles and boulders. However, since drilling logs do not indicate a significant increase in the volume of boulders with increasing depth, the higher velocities are more probably due to the relatively dense nature of the deposit. Near the edges of the canyon bottom and overlying the bedrock downstream of the left abutment, the colluvium/alluvium is characterized by compressional wave velocities intermediate between the saturated and unsaturated alluvium (line 85-3). Since these deposits are dry, the velocities are higher than that for dry streambed alluvium because of the denser, slightly silty sand matrix. A summary of refractive seismic surveys in the streambed is shown in table A-16.

Table A-16. Results of Refractive Seismic Surveys in Foundation Alluvium (lines 84-1 to 84-5).

Layer	Velocity (ft/s)	Depth to Base (ft)	Thickness (ft)	Inferred Geologic Interpretation
1	500-1,400	2-13	2-13	Unsaturated streambed alluvium
2	1,800-3,000	11-29	9-18	Colluvium/alluvium near canyon edge
	1,900	65-85	60-80	Colluvium/alluvium d/s from outlet portal
3	6,000-8,000	35-100	30-80	Saturated streambed alluvium
4	9,800-12,000	n.a.	n.a.	Bedrock (buried at depth
	3,500-6,700	n.a.	n.a.	Bedrock (surface exposures)

Crosshole and Downhole Geophysical Surveys

7-12 As with the refractive seismic surveys, the crosshole and downhole geophysical surveys indicate that the streambed alluvium is loose to moderately dense near the surface and dense to very dense below a depth of about 20 feet. Downhole and crosshole shear wave velocities average 750 and 770 ft/s, respectively, for the surficial soils above the groundwater, and 2,000 and 1,600 ft/s, respectively, for the saturated sands and gravels below groundwater. The compressional wave (P-wave) velocities average 1,520 ft/s in the surficial soils and increase with depth to 7,500 ft/s in the saturated river sands and gravels. See attachment A-3 for the plots of shear wave and compressional wave velocities prepared by Woodward-Clyde Consultants. These values are very similar to the results of crosshole seismic surveys at the previously investigated Mentone damsite, averaging slightly higher at the Seven Oaks damsite.

Large-Scale Density Tests

7-13 Ten large-scale density tests were conducted under contract by Woodward-Clyde Consultants at approximately 5-foot depth increments in the 3 trenches excavated in the foundation in October 1984. A fiberglass ring, 8 inches high and 4 feet in diameter, was used to obtain in-place densities of the alluvial materials. The area to be tested was hand leveled to provide a relatively smooth surface upon which to set the ring so that it did not rock or shift over high points. Plastic sheeting was then placed over the ring and anchored at the base. Water was placed in the ring to a fixed level to determine the initial volume of the ring. After removing the water and plastic, materials within the ring were excavated with picks and shovels to a depth of approximately 1.5 feet. Between 800 and 1,300 pounds were removed for each test. The plastic sheeting was replaced inside the ring and filled with water to determine the final volume of the hole. The volume of the test hole was determined by calculating the difference in volume of water required to fill the ring before and after excavation. The density of the in-place material was calculated by dividing the weight of the excavated material by the volume of the hole. The materials removed in the tests were placed in 55-gallon drums and retained for classification and detailed laboratory tests.

7-14 The dry densities of the in-place near surface materials, shown on plate A-51, ranged from 109 to 137 lb/ft^3, with an average value of 125 lb/ft^3. Based on experience with similar materials in the general vicinity of the site, these values are less uniform and lower than expected. For example, the average in-place dry density of 33 large-scale density tests in the foundation of the proposed Mentone Dam site, about 4 miles downstream where somewhat less rock is present, was 133 lb/ft^3. At the proposed Lytle Creek damsite, where depositional and material characteristics are very similar, the average of 7 large-scale density tests was 136 lb/ft^3. All tests were conducted with the same laboratory equipment and procedure and, the materials being similar in size, origin, depositional nature, and density, the results of field and laboratory tests would be expected to be similar.

7-15 Several possible reasons for the wide range and generally low values are apparent: physical errors in testing and measurement, selective excavation of field density tests in intervals of relatively sandy materials that would be easier to dig, improper sampling of materials for laboratory testing, poor reporting of field conditions, or actual existence of highly variable conditions in the foundation. Extensive foundation investigations are planned during the FDM study phase to resolve the apparent discrepancies and form a basis for design of the embankment foundation.

Aquifer Test

7-16 A drawdown test was performed during November 1984 to determine the aquifer characteristics of the foundation alluvium. Water was pumped out of a 10-inch diameter well (boring B84-9, shown on plate A-9) at a rate of 870 gal/min while the amount of drawdown was measured in 6 observation wells. The distance to the observation wells ranged from 15 to 428 feet from the pumping well. After pumping, the rate of recovery of the water level was measured in each well.

7-17 The data from the pump test were analyzed assuming an average depth of alluvium of 80 feet, an average thickness of the aquifer of 65 feet, and that the bedrock canyon walls act as impermeable boundaries to the unconfined alluvial aquifer. The Santa Ana River was flowing at a rate of approximately 3 ft^3/s at a distance of about 150 feet from the pumping well.

7-18 The results of the aquifer test indicate that the permeability, or hydraulic conductivity, of the streambed alluvium ranges from about 1,900 to 2,700 $gal/day/ft^2$. The average permeability is about 2,220 $gal/day/ft^2$, or about 300 ft/day (0.10 cm/sec). Based on assumed aquifer thickness, the average transmissivity of the aquifer was calculated to be 133,000 $gal/day/ft^2$ with a storativity value of about 0.015.

LABORATORY TESTS AND RESULTS

7-19 Laboratory tests were conducted as described in the following paragraphs to determine the gradation, density and strength characteristics of the streambed alluvium.

Mechanical Analysis

7-20 The gradation of each large-scale density sample was determined by oven drying the materials and then shaking them through screens with openings ranging from 3 inches to 0.074 mm in accordance with Appendix V, EM 1110-2-1906, "Laboratory Soils Testing." Atterberg limits tests and hydrometer analysis were not performed because of the obvious nonplastic nature of the materials. The materials were classified in accordance with ASTM D 2487. The soil logs with the results of the mechanical analysis of the minus 3 inch portion of the materials and the classifications are shown on plate A-50. Gradation curves are shown on plate A-51.

7-21 The limited investigation and testing to date indicate that the streambed alluvium in the vicinity of the proposed dam is predominantly gravelly sand with varying amounts of cobbles and boulders. Based on the samples obtained from the large-scale density tests, the minus 6-inch materials typically contain less than 2 percent nonplastic fines, about 40 percent sand, and about 60 percent larger than No. 4 size. The gradation range of the samples is shown on plate A-51 along with the average and upper and lower quartiles. By visual estimates, approximately one-third of the alluvial foundation material consists of cobbles and boulders up to 5 feet in diameter. Inspection of the test trenches indicates that point-to-point contact between cobbles and boulders is relatively common, however, the space between these larger stones is filled with sand and gravel. No evidence was noted of nesting of cobbles or of seams of open-graded gravels. Occasional pockets of poorly-graded, medium-grained sand were observed in the sides of the trenches. Further investigation indicated, however, that the sand pockets were relatively small, averaging 4 to 8 feet in length, and discontinuous. The average thickness of the sand layers was about 1 foot.

Maximum Density

7-22 The maximum density of each large-scale in-place density sample was determined using vibratory compaction methods on the minus 6-inch, minus 3-inch, and minus 1-1/2-inch portions of the sample. The test procedure and equipment used to determine the maximum density of the minus 6-inch and minus 3-inch portions was developed by the California Department of Water Resources to obtain maximum densities of coarse grained cohesionless soils. Approximately 500 pounds of minus 3-inch material were first placed in a 27-inch diameter by 30-inch high mold. A calculated amount of water to almost saturate the sample was added prior to placement, the surface leveled, and a steel surface plate placed on top of the sample. A 1,060 lb concrete block surcharge, equivalent to about 2 lb/in^2, was then placed on the surface plate, the entire mold hoisted off the ground, and the mold vibrated at approximately 7,500 vib/min for exactly 15 minutes. Readings were taken at 4 locations after 1, 5, 10 and 15 minutes. The readings consisted of measuring from the top of the mold to the top of the steel plate. The volume of the sample was determined by subtracting the averaged readings and plate thickness from the mold height and multiplying the sample height times the mold area. Any pumped material was removed and the weight of the dry pumped material was determined. The dry density was determined by dividing the volume into the dry weight of the material placed in the mold minus the dry weight of the pumped material.

7-23 The sample was removed from the mold, and an appropriate amount of minus 3-inch material was removed and replaced with 3- to 6-inch material to produce another 500 pounds sample whose gradation was then representative of the minus 6-inch portion of the entire sample. This material was tested in the 27-inch diameter mold to determine the maximum density of this portion of the material.

7-24 The maximum and minimum densities of the minus 1-1/2-inch portions of the field density samples were determined using ASTM D 4253 and D 4254 test methods. These tests utilize an 11-inch diameter, 0.5 ft^3

mold, vibrated for 8 minutes at 3,600 cycles/min with a 2 lb/in^2 surcharge. These materials were tested dry, as recommended in Appendix XII, EM 1110-2-1906, "Laboratory Soils Testing."

7-25 Rock correction formulas were applied to all the maximum density values to compensate for the varying amounts of oversize material that were not included in the laboratory tests due to equipment limitations. The oversize material was replaced mathematically to adjust the laboratory density to the in situ density by the following equation:

$$T = \frac{1}{\frac{P}{62.4G} + \frac{1-P}{M}}$$

T = Adjusted dry density
P = Decimal percent of oversize
G = Bulk specific gravity of oversize
M = Dry density of minus fraction used in density test

Since manipulation of the amount of oversize material also changes the void ratio of the sample, the effects of disturbance were also investigated. Based on suggested corrections in published literature, it was found that adjustments due to disturbance were usually less than one percent for the materials at Seven Oaks Dam and could be ignored. However, additional evaluation of disturbance effects will be conducted during the FDM study phase.

7-26 Laboratory maximum density test results on the minus 6-inch, minus 3-inch, and minus 1-1/2-inch portions of the materials from the large-scale field density tests are summarized in table A-17. This shows that laboratory maximum densities ranged from 112 to 130 lb/ft^3 for minus 6-inch materials, from 93 to 131 lb/ft^3 for minus 3-inch materials, and from 107 to 135 lb/ft^3 for minus 1-1/2-inch materials. When the densities are adjusted for the amount of oversize material that was not included in the test, the maximum density values ranged from 119 to 138 lb/ft^3, from 93 to 143 lb/ft^3, and from 107 to 145 lb/ft^3 for minus 6-inch, minus 3-inch, and minus 1-1/2-inch materials, respectively. These numbers may not accurately represent the actual conditions, however, since some of the in situ density samples are in question, the number of samples is low, and the percent of oversize material corrected for ranged up to 51 percent, whereas 30 percent is the commonly accepted upper limit for applying rock corrections without introducing significant error.

7-27 When compared with these adjusted laboratory maximum densities, the in situ densities correspond to relative compactions ranging from 83 to 103 percent for minus 6-inch materials, from 83 to 117 percent for minus 3-inch materials, and from 80 to 104 percent for minus 1-1/2-inch materials. The corrected maximum densities and the in-place dry density for each field density test are plotted versus depth on plate A-51 for each trench.

Table A-17: Laboratory Maximum Densities of Foundation Alluvium

Test Trench No.	Density Test No.	Depth (feet)	Max. Size Lab. Test (inches)	Mold Size (inches)	Percent Oversize	Lab. Max. Density (lb/ft3)	Adj. Max. Density (lb/ft3)	Field Density (lb/ft3)	Relative Compaction (percent)
TT 84-1	1	0.5	6	27	0	124	124	118	95
			3	27	7	121	123		96
			1.5	11	18	127	132		89
	2	4.2	6	27	0	130	130	123	95
			3	27	12	125	128		96
			1.5	11	24	131	137		89
	3	10.0	6	27	8	130	132	133	100
			3	27	11	129	133		100
			1.5	11	27	135	142		93
	4	14.0	6	27	26	130	137	114	83
			3	27	27	129	137		83
			1.5	11	37	130	141		80
	5	19.5	6	27	34	127	138	136	98
			3	27	39	131	143		94
			1.5	11	51	120	139		97
TT 84-2	1	2.0	6	27	0	126	126	130	103
			3	27	1	119	120		109
			1.5	11	6	124	125		104
	2	6.8	6	27	0	-	-	109	-
			3	27	1	93	93		117
			1.5	11	1	107	107		102
	3	13.0	6	27	35	119	132	136	103
			3	27	40	116	132		103
			1.5	11	47	119	137		98
TT 84-3	1	2.6	6	27	33	126	138	137	99
			3	27	36	125	138		99
			1.5	11	48	129	145		94
	2	7.8	6	27	17	112	119	112	94
			3	27	19	109	117		95
			1.5	11	26	113	123		91

Triaxial Compression

7-28 Two samples of foundation material from the in-place density tests were selected as representative of the relatively lower and higher sand contents of the foundation gradation range. These samples, about 1,200 lb each, were tested in a 12-inch diameter triaxial compression apparatus under consolidated undrained conditions to determine the shear strengths of the foundation materials. During the tests, the pore pressures were monitored in order to calculate the consolidated drained strengths also. For each of the two types of material, tests were run at confining pressures of 1, 2, and 4 ton/ft^2 on minus 2-1/2-inch material. The results are shown on plate A-52 in the form of three Mohr's circles for each material type.

Government Canyon Ridge

MAPPING

7-29 The rock comprising Government Canyon Ridge in the area of the proposed excavation is composed entirely of moderately to highly weathered and highly fractured gneiss (gn). The rock easily separates along most discontinuities and can be broken with a light to moderate hammer blow. Mineral foliation trends N80E to N70W and dips steeply to the south. A 5-foot wide quartzo-feldspathic dike, cutting diagonally through the ridge can be observed along the south slope of the ridge. The dike is brittle and appears to be only slightly more resistant than the gneiss. A vertical fault was mapped at the southwest end of the proposed excavation within a large drainage, though the direction of movement could not be discerned. Colluvial cover appears to be 5 to 10 feet thick on the lower slope on the north side, and thin everywhere else in the vicinity.

DRILLING SUMMARY

7-30 Logs of holes drilled on Government Canyon Ridge are presented on plates A-24 and A-25. An overall summary of drilling on Government Canyon Ridge is presented in table A-18.

Table A-18. Government Canyon Ridge, Drilling Summary.

Hole No.	Rocktype	Bedrock Depth (ft)	Recovery (%)	RQD (%)	Discon. Freq. (per ft)	Point Load (lb/in^2)	K (ft/day)
C86-09	Gneiss	0.0	77	22	5.8	158	-
C86-11	Gneiss	0.0	85	28	5.0	214	-
C86-14	Gneiss	0.0	89	34	6.8	181	2.50
C86-27	Gneiss	0.0	93	41	4.8	154	-
Average:		0.0	86	31	5.6	177	-

GEOPHYSICAL SURVEY RESULTS

7-31 The results of geophysical surveys conducted at Government Canyon Ridge are presented in the following paragraphs.

Refractive Seismic

7-32 The time-distance graph and subsurface interpretation for the refractive seismic line conducted on Government Canyon Ridge, 86-10, is presented on plate A-38. Basically, two separate velocity zones are identifiable in the bedrock. Layer 1 (V1) probably represents most highly weathered and fractured rock while layer 2 (V2) is somewhat better quality rock. Both velocity zones, however, are representative of highly fractured and weathered gneissic bedrock. The measured velocities and calculated depth for the layers identified are presented in table A-19.

Table A-19. Government Canyon Ridge, Results of Refractive Seismic Survey.

Line No.	V1 (ft/s)	Depth (ft)	V2 (ft/s)
86-10	1950	30	4275

Downhole Seismic

7-33 Downhole geophysical surveys were conducted in holes C86-14 and C86-27. Apparent P-wave velocities measured over 10-foot intervals ranged from 3,077 to 10,000 ft/s with the overall average velocity for the holes approximately 5,000 ft/s. Although apparent interval velocities varied greatly, it was possible, as shown on plate A-40, to distinguish general velocity differences with depth as measured in each hole. The results are presented in table A-20.

Table A-20. Government Canyon Ridge, Results of Downhole Geophysical Surveys.

Hole No.	Depth Interval (ft)	Average Velocity (ft/s)
C86-14	0-50	3,410
C86-14	50-180	7,165
C86-27	0-20	2,000
C86-27	20-60	5,000
C86-27	60-120	8,000

LABORATORY TEST RESULTS

7-34 The results of laboratory tests conducted on samples from Government Canyon Ridge are presented in the following paragraphs.

Unconfined Compressive Strength

7-35 Unconfined compression tests were conducted on 2 samples from holes on Government Canyon Ridge. The results of those tests are presented below.

Table A-21. Government Canyon Ridge, Results of Compressive Strength Tests.

Hole No.	Rocktype	Depth (ft)	Unconfined Compressive Strength (lb/in^2)
C86-09	Gneiss	121.2	2,220
C86-11	Gneiss	184.3	5,337

Rock Quality

7-36 Five boxes of HQ-size core were tested for the rock quality parameters previously described in Section VI, Laboratory Testing Procedure - Rock Quality. The results of those tests are presented in table A-22 along with the average of all point load strengths measured for core in the interval tested.

Table A-22. Government Canyon Ridge, Results of Rock Quality Tests.

Hole No.	Test Interval (ft)	Point Load (lb/in^2)	LA Loss (%)	Bulk SSD	Absorption (%)	Wet/Dry
C86-09	73.3-85.5	114	84.3	2.60	2.73	passed
C86-11	50.8-59.5	134	85.2	2.59	2.83	passed
C86-11	179.0-188.0	164	75.9	2.77	1.39	passed
C86-14	170.3-180.8	130	78.1	2.67	1.64	passed
C86-27	99.0-111.0	133	82.1	2.50	2.84	failed
Average:		135	81.1	2.63	2.29	

Petrographic Analysis

7-37 Although the rockmass in Government Canyon Ridge is mapped as gneiss because of its banded and heterogeneous appearance, the petrographic analyses of the samples tested indicated the rocks are more appropriately classified as igneous, not metamorphic. Principal mineral constituents include plagioclase feldspar, quartz, orthoclase feldspar, biotite and hornblende. Major accessory minerals are magnetite, sphene and zircon. Samples were classified as slightly to moderately weathered, closely fractured with minor to intense evidence of compressional stress. Based upon the Colorado School of Mines Classification of Igneous Rock, all samples classified as either monzonite or quartz monzonite. (This petrographic rock classification name is not to be confused with the quartz monzonite rock terrane (qm) upstream from the North Branch fault zone). The sample from hole C86-09 showed a trace of montmorillonite. The low concentration of clay minerals in most samples made them difficult to detect by x-ray diffraction in whole rock analysis. A summary of rock classification data including the ratio of plagioclase to orthoclase feldspar, and percentage of total quartz and mafic minerals is presented in table A-23.

Table A-23. Government Canyon Ridge, Results of Petrographic Analyses.

Hole No.	Depth (ft)	Plagioclase/ Orthoclase	Quartz (%)	Mafics (%)	Rock Name
C86-09	73	37/63	9	10	monzonite
C86-11	55	65/35	9	10	monzonite
C86-11	184	63/37	15	20	qtz. monzonite
C86-14	174	53/47	20	15	qtz. monzonite
C86-27	111	44/56	20	10	qtz. monzonite
Average:		52/48	15	12	qtz. monzonite

Spillway

MAPPING

7-38 The topographic relief in the spillway area is rugged with 3V on 4H slopes on the north side of the saddle and steeper 3V on 4H to 5V on 4H slopes on the south side. Thin colluvial cover, up to three feet thick, and heavy vegetation effectively conceal the upstream side of the spillway saddle. There is very little colluvial cover on the downstream side. Most of the spillway excavation will be in gneiss. Towards the extreme north end of the spillway site, and trending semi-perpendicular

to the proposed centerline, fault "A" (see discussion in faulting) juxtaposes gneissic (gn) rock to the south with diorite (qd) to the north. At the surface the gneiss is highly to completely weathered and highly fractured. Mineral foliation trends approximately N65E and dips steeply to the southwest. The gneiss can be easily separated along discontinuities and many samples can be broken by hand. Mafic-rich inclusions in gneiss were observed throughout the damsite, but one large band was mapped in the spillway because of its size (100 feet wide) and continuity. It is seen to the east within the spillway site and is traceable to the west 1,000 feet from the spillway centerline before it is covered by colluvium. Farther west, scattered exposures suggest that this dark-colored zone continues into Santa Ana River canyon. The rock in this zone is softer than the rest of the gneiss because of the higher concentration of biotite. As shown on plate A-6, three notable faults have been mapped in the spillway site, fault "A", as previously discussed, and two steeply dipping faults observed on the south side of the spillway saddle. These faults trend nearly parallel to the proposed conterline and appear to offset the mafic-rich zone by up to 300 feet.

DRILLING SUMMARY

7-39 Logs of holes drilled in the spillway are presented on plates A-26 through A-29. A summary of drilling conducted in the spillway is presented in table A-24. Also shown in table A-24 are the average values for: (1) all holes drilled in the spillway, (2) all spillway holes drilled in gneiss, and (3) all holes exclusive of C86-15, C86-17 and C86-22 which contained markedly inferior quality rock which is judged to be not representative of the spillway rockmass as a whole.

Table A-24. Spillway Drilling Summary.

Hole No.	Rocktype	Bedrock Depth (ft)	Recovery (%)	RQD (%)	Discon. Freq. (per ft)	Point Load (lb/in^2)	K (ft/day)
C86-15	Gneiss	0.0	37	4	6.7	-	-
C86-16	Gneiss	0.0	97	49	6.3	222	-
C86-17	Gneiss	0.0	69	9	4.8	43	-
C86-18	Gneiss	0.0	93	40	6.2	147	-
C86-19	Gneiss	0.0	75	24	5.9	146	-
C86-20	Gneiss	0.0	95	40	7.1	127	-
C86-21	Gneiss	0.0	96	40	5.8	169	-
C86-22	Diorite	5.0	84	27	5.3	76	-
Average, all holes:			81	29	6.0	133	-
Average, gneiss:			80	29	6.1	142	-
Average exclusive of C86-15, 17 and 22:			91	39	6.3	162	-

GEOPHYSICAL SURVEY RESULTS

7-40 The results of geophysical surveys conducted at the spillway are presented in the following paragraphs.

Refractive Seismic

7-41 The time-distance graphs and subsurface interpretations for the refractive seismic lines conducted in the spillway are presented on plates A-38 and A-39. Although as many as four separate velocity zones are identifiable, they all represent variability within the gneissic rockmass, both vertically and laterally. The measured velocities (V) and approximate interface depths (D) are presented in table A-25.

Table A-25. Spillway, Results of Refractive Seismic Surveys.

Line No.	V1 (ft/s)	D1 (ft)	V2 (ft/s)	D2 (ft)	V3 (ft/s)	D3 (ft)	V4 (ft/s)
86-11	1,450	5	2,550	25	4,200	50?	11,000
87-01	1,400	10	4,600	30	6,500		

Downhole Seismic

7-42 Downhole geophysical surveys were conducted in holes C86-19, C86-20 and C86-22. Apparent P-wave velocities over 10-foot intervals ranged from 1,429 to 15,385 ft/s with the average velocity for the all the holes approximately 3,500 ft/s. Although apparent interval velocities varied greatly, it was possible to distinguish general velocity differences with depth as measured in each hole. Time-distance graphs are presented on plate A-40 and the averaged results are presented in table A-26.

Table A-26. Spillway, Results of Downhole Geophysical Surveys.

Hole No.	Depth Interval (ft)	Average Velocity (ft/s)
C86-19	0-20	1,800
C86-19	20-40	2,500
C86-19	40-140	6,150
C86-20	0-20	1,500
C86-20	20-100	3,900
C86-22	0-25	1,700
C86-22	25-50	3,600
C86-22	50-110	6,500

LABORATORY TEST RESULTS

7-43 The results of laboratory tests conducted on samples from the spillway are presented in the following paragraphs.

Unconfined Compressive Strength

7-44 Unconfined compression tests were conducted on 5 core samples from the spillway. The results of those tests are presented in table A-27. If a point load test was conducted on one end of a core piece sent to the laboratory, that value and the ratio of compressive strength to point load index strength is also presented. Compressive strength values with an asterisk (*) are of questionable validity because of failure along veins or foliation.

Table A-27. Spillway, Results of Compressive Strength Tests.

Hole No.	Rocktype	Depth (ft)	Unconfined Compressive Strength (lb/in^2)	Point Load Index (lb/in^2)	Ratio
C86-16	Gneiss	72.1	3,350*	-	-
C86-16	Gneiss	144.5	2,300*	205	11.2
C86-18	Gneiss	302.8	8,206	161	51.0
C86-19	Gneiss	179.5	6,640*	178	37.3
C86-21	Gneiss	81.4	7,449	-	-
Average:			5,589*		33.2

Rock Quality

7-45 Thirteen boxes of HQ-size core were tested for the rock quality parameters previously described in Section VI, Laboratory Testing Procedure - Rock Quality. The results of those tests are presented in table A-28 along with the average of all point load strengths measured for core in the interval tested.

Table A-28. Spillway, Results of Rock Quality Tests.

Hole No.	Test Interval (ft)	Point Load (lb/in^2)	LA Loss (%)	Bulk SSD	Absorption (%)	Wet/Dry
C86-16	77.4-87.6	168	70.7	2.57	2.17	passed
C86-16	246.6-257.1	160	58.5	2.56	1.59	passed
C86-17	104.4-116.0	27	89.9	2.54	3.99	failed
C86-18	103.4-114.1	135	80.2	2.57	2.25	passed
C86-18	267.4-278.2	133	77.9	2.57	3.04	passed
C86-19	61.8-72.7	86	74.3	2.44	2.71	failed
C86-19	140.7-152.0	116	79.0	2.52	2.29	passed
C86-20	41.1-52.1	153	75.0	2.56	2.52	passed
C86-20	192.0-202.5	120	88.0	2.54	2.22	failed
C86-21	47.1-57.8	120	78.5	2.51	3.76	failed
C86-21	120.6-128.0	152	68.4	2.64	2.49	passed
C86-22	0.0-32.8	56	91.7	2.48	4.74	failed
C86-22	69.2-80.2	89	87.9	2.61	2.72	passed
Average:		117	78.5	2.55	2.81	

Petrographic Analysis

7-46 Although most all of the rockmass in the spillway is mapped as gneiss because of its banded and heterogeneous appearance, the petrographic analyses of the samples tested indicated the rock is more appropriately classified as igneous, not metamorphic. Principal mineral constituents include plagioclase feldspar, quartz, orthoclase feldspar, biotite and hornblende. Major accessary minerals are magnetite, sphene and zircon. Samples were classified as slightly to highly weathered and closely fractured with minor to extreme evidence of compressional stress. Based upon the Colorado School of Mines Classification of Igneous Rock, the samples varied from granite to diorite with 13 of the 15 samples tested classifying as granodiorite or diorite. Half of the samples analyzed showed traces of montmorillonite as a product of weathering. A summary of rock classification data including the ratio of plagioclase to orthoclase feldspar, and percentage of total quartz and mafic minerals is presented below.

Table A-29. Spillway, Results of Petrographic Analyses.

Hole No.	Depth (ft)	Plagioclase/ Orthoclase	Quartz (%)	Mafics (%)	Rock Name
C86-16	73	69/31	8	25	diorite
C86-16	78	67/33	10	9	diorite
C86-16	249	22/78	22	5	granite
C86-17	107	78/22	20	5	granodiorite
C86-18	107	17/83	20	1	granite
C86-18	278	67/33	3	20	diorite
C86-19	67	75/25	25	10	granodiorite
C86-19	141	76/24	20	15	granodiorite
C86-19	180	79/21	15	30	granodiorite
C86-20	46	73/27	30	10	granodiorite
C86-20	197	74/26	15	5	granodiorite
C86-21	47	70/30	10	5	diorite
C86-21	126	67/33	15	60	granodiorite
C86-22	27	68/32	5	15	diorite
C86-22	111	44/56	15	15	granodiorite
Average:		64/36	16	15	granodiorite

Right Abutment

MAPPING

7-47 Within the abutment excavation area, the slopes are locally steeper than 1V on 1H especially at the lower elevations. The slopes at the higher elevations flatten to 1V on 2H and are covered by colluvium. The colluvium, which averages an estimated 5 feet in thickness, covers about half of the area. Terrace deposits (Qt) are exposed intermittently near the 2,400-foot elevation contour. The maximum thickness is estimated to be about 30 feet with very limited areal extent. The clasts are primarily boulder sized to as much as six feet in diameter or more with the rest as gravel and cobble-sized, all in a sand-silt matrix. The clasts are composed of granite, diorite, monzonite and some gneiss and quartzite. This material is likely to be encountered, during excavation of the abutment, on the downstream side of the centerline. The bedrock at the right (west) abutment of the proposed damsite is composed of diorite and gneiss. An apparently gradational contact between the two rockmasses occurs approximately under the centerline of the proposed dam with the diorite on the downstream side. The bedrock is highly to completely weathered wherever encountered, except in the tributary drainage canyon just upstream from centerline. This canyon has cut 250 to 300 feet into the ridge, exposing resistant gneiss that

forms near vertical cliff faces in the drainage bottom and side walls. Mineral foliation in the gneiss trends between N75W and N80E and dips steeply to both the north and south. Prominent joint sets in the diorite are N5W, 85NE-SW and N35W, 45SW. One vertical joint set observed in the gneiss is oriented north-south.

DRILLING SUMMARY

7-48 Logs of holes drilled at the right abutment are presented on plates A-19 and A-20. A summary of drilling at the right abutment is presented in table A-30. Holes C86-01 and C86-03, although drilled in the streambed, are included with the right abutment drilling summary because their primary purpose was to explore the covered toe of the abutment rock slope.

Table A-30. Right Abutment, Drilling Summary.

Hole No.	Rocktype	Bedrock Depth (ft)	Recovery (%)	RQD (%)	Discon. Freq. (per ft)	Point Load (lb/in^2)	K (ft/day)
C84-07	Gneiss	21.0	67	18	7.6	174	0.10
C84-08	Diorite	0.0	88	45	5.4	221	0.10
C86-01	Diorite	17.5	90	40	4.4	169	0.01
C86-03	Diorite	28.0	99	52	3.6	145	0.01
C86-23	Gneiss	0.0	90	33	3.4	351	0.08
Average, all rock:			87	38	4.9	212	0.05
Average, diorite:			92	46	4.5	178	0.04
Average, gneiss:			79	26	5.5	263	0.08

GEOPHYSICAL SURVEY RESULTS

7-49 Neither refractive nor downhole geophysical surveys were conducted at the right abutment.

LABORATORY TEST RESULTS

7-50 The results of laboratory tests conducted on samples from the right abutment are presented in the following paragraphs.

Unconfined Compressive Strength

7-51 Four unconfined compression tests were conducted on 3 samples from holes at the right abutment. The results of those tests are presented in table A-31.

Table A-31. Right Abutment, Results of Compressive Strength Tests.

Hole No.	Rocktype	Depth (ft)	Unconfined Compressive Strength (lb/in^2)
C84-08	Diorite	65.2	2,540
C84-08	Diorite	65.6	1,790
C86-01	Diorite	71.8	5,290
C86-03	Diorite	65.2	10,250
Average:			4,968

Rock Quality

7-52 Rock quality tests were not conducted on core samples from the right abutment.

Petrographic Analysis

7-53 Petrographic analyses were not conducted on core samples from the right abutment.

Left Abutment

MAPPING

7-54 Much of the abutment site is mantled with colluvium and moderately heavy scrub vegetative cover, while bedrock is exposed along the ridges and knobs. Colluvial cover is about 6 feet thick on the upper part of the abutment site becoming as thick as 116 feet, and perhaps thicker, towards the bottom. The extensive colluvial cover on the abutment accounts for the more subdued relief where slopes seldom exceed 1V on 1.5H. No terrace deposits were observed within the vicinity of the left abutment. None were encountered during construction of the spillway access road nor were they encountered in core hole, C86-04. Diorite forms much of the left (east) abutment at the damsite. A body of gneiss is exposed at the upstream end of the abutment, where it is in sharp contact with the diorite to the south. Scattered exposures of gneiss were observed near the centerline of the abutment, perhaps as roof pendants, remnants of country rock (gn) intruded and surrounded by the younger dioritic rock. The diorite is slightly to highly weathered and highly fractured making it easy to unravel along discontinuities, but requiring hammer blows to break. Joints in the diorite have a wide variety of orientations. The prominent trends were N50W and N45E with a

wide range of dips to the east. One joint set was oriented N20E, 85SE. The few joints that could be identified in the gneiss are oriented N30W, 60SW and N-S, 25E. A fault observed in the road cut, at the base of the abutment just downstream from centerline, is a 12-foot wide zone of highly sheared diorite (qd) that could only be traced a short distance laterally. There is no evidence to suggest that this fault is active or potentially active.

DRILLING SUMMARY

7-55 At the left abutment, both bucket auger and core drilling were used to explore the subsurface. The results of both drilling operations are presented in the following paragraphs.

Core Drilling

7-56 Logs of the two core holes drilled at the left abutment are presented on plate A-18. A summary of drilling at the left abutment is presented in table A-32.

Table A-32. Left Abutment, Core Drilling Summary.

Hole No.	Rocktype	Bedrock Depth (ft)	Recovery (%)	RQD (%)	Discon. Freq. (per ft)	Point Load (lb/in^2)	K (ft/day)
C84-06	Diorite	31.7	93	34	5.8	320	0.05
C86-04	Diorite	116.0	78	18	6.4	148	0.04
Average:			86	26	6.1	234	0.05

Bucket Auger Drilling

7-57 In addition to collecting samples of overburden materials on the left abutment for laboratory testing, information from bucket auger drilling was used to determine the depth to bedrock where possible. The bedrock depths, based primarily on drilling action, and the approximate elevation of the holes is presented in table A-33. Bedrock was not encountered in hole LAH-4 because a large rock caved into the hole at the 35-foot depth and the hole could not be advanced. The soil logs of the bucket auger holes are presented on plate A-53.

Table A-33. Left Abutment, Bucket Auger Drilling Summary.

Hole No.	Surface Elevation (ft)	Bedrock Depth (ft)
LAH-1	2215	90
LAH-2	2280	47
LAH-3	2265	44
LAH-4	2215	35+

GEOPHYSICAL SURVEY RESULTS

7-58 The results of geophysical surveys conducted at the left abutment are presented in the following paragraphs.

Refractive Seismic

7-59 The time-distance graphs and subsurface interpretations for the two refractive seismic lines conducted at the left abutment are presented on plates A-37 and A-38. Line 85-4, at the toe of the abutment at centerline, was conducted to estimate the depth of overburden and line 86-09 was conducted on the abutment slope downstream from the centerline to estimate the amount of colluvial cover which will require excavation. The measured velocities (V) and generalized depths (D) are presented in table A-34. An analysis of the results is presented in Section VIII - Analysis of Data.

Table A-34. Left Abutment, Results of Refractive Seismic Surveys.

Line No.	V1 (ft/s)	D1 (ft)	V2 (ft/s)	D2 (ft)	V3 (ft/s)
85-4	1,800	5	6,000-7,000	35	12,000
86-09	1,950	30	4,950		

Downhole Seismic

7-60 Downhole geophysical surveys were not conducted in either of the core holes at the left abutment.

LABORATORY ROCK TEST RESULTS

7-61 The results of laboratory tests conducted on rock samples from the left abutment are presented in the following paragraphs.

Unconfined Compressive Strength

7-62 The results of unconfined compressive strength tests conducted on 4 core samples from the left abutment are presented in table A-35. If a point load test was conducted on one end of a core piece sent to the laboratory, that value and the ratio of compressive strength to point load index strength is also presented.

Table A-35. Left Abutment, Results of Compressive Strength Tests.

Hole No.	Rocktype	Depth (ft)	Unconfined Compressive Strength (lb/in^2)	Point Load Index (lb/in^2)	Ratio
C84-06	Diorite	133.0	13,180	-	-
C86-04	Diorite	131.5	2,460	-	-
C86-04	Diorite	275.8	3,817	227	16.8
C86-04	Diorite	367.5	5,106	-	-
Average:			6,141		

Rock Quality

7-63 Rock quality tests were not conducted on core from the left abutment.

Petrographic Analysis

7-64 A sample from hole C86-04 was sent to the South Pacific Division laboratory for petrographic analysis. The sample classified as diorite with the principal constituents being plagioclase feldspar, quartz, orthoclase feldspar, biotite and hornblende. The sample was moderately to highly weathered and intensely fractured as the result of severe compressional stress. A summary of rock classification data including the ratio of plagioclase to orthoclase feldspar, and percentage of total quartz and mafic minerals is presented in table A-36.

Table A-36. Left Abutment, Results of Petrographic Analyses.

Hole No.	Depth (ft)	Plagioclase/ Orthoclase	Quartz (%)	Mafics (%)	Rock Name
C86-04	132	74/26	10	10	diorite

LABORATORY SOIL TESTS AND RESULTS

Soil Classification Tests

7-65 Mechanical analysis and Atterberg limits tests were conducted on all soil samples collected from the bucket auger test holes in the left abutment. The results of these tests were used to classify the materials in accordance with ASTM test method D 2487, and are shown on the soil logs on plate A-53. These tests indicate that the colluvial soils are predominantly unconsolidated noncohesive silty gravelly sands with weathered, angular rock fragments up to 12 inches in diameter.

Moisture-Density Relationships

7-66 Five-point compaction tests were conducted on selected large samples in accordance with ASTM C 698 test methods to determine the moisture-density relationships of the left abutment overburden soils. The results, shown on plate A-54, indicate that the maximum density ranges from 126 to 132 lb/ft^3, and that the optimum moisture content is between 7 and 11 percent. Maximum densities determined in a vibrated 11-inch mold with a surcharge weight (ASTM D 4253) varied from 119 to 132 lb/ft^3. Results of laboratory tests on materials from the left abutment are shown graphically on plate A-54.

Outlet Works

MAPPING

7-67 The outlet works will probably be tunneled exclusively through diorite. An inspection of the Edison tunnel, located within the same hill as the proposed tunnel, see plate A-9, allowed mapping of what should be similar geological conditions. This water conveyance tunnel is approximately 7 feet in diameter and concrete lined on the bottom and sides with a free-standing rock crown for approximately 900 feet of its 1,350 foot total length. The Edison tunnel is 1,000 feet to the southwest of the proposed tunnel, 250 feet higher in elevation and similarly aligned. Most of the Edison tunnel is in diorite. However, because the outlet portal is slightly south of the end of the proposed outlet works tunnel, the Edison tunnel does intersect fault "A" and a

portion of the gneissic rockmass. Fault "A" (N85W,75SW) was observed as a six-inch wide, healed and competent gouge zone. There is no sign of caving and no tunnel support where the fault crosses the tunnel alignment. The inspection of the remainder of the tunnel revealed a hard, slightly to moderately weathered, and complexly jointed and faulted diorite. The spacing and the variable orientation of the joint sets results in a blocky rockmass. The joints were generally dry; a few had minor seepage, particularly behind a 180-foot long section of full concrete lining 400 feet from the southwest end. Minor shears and gouge were observed along the length of the tunnel. The nature of the rockmass where the crown is concrete lined is unknown. The support may be necessary because of significant shear zones or particularly adverse joint orientations that affect rockmass stability.

7-68 The outlet portal area of the proposed outlet works tunnel is completely covered by colluvium and was therefore not mapped. The steep rockface above the inlet portal, on the other hand, is fully exposed. The diorite rockmass at this location is relatively fresh in appearance and blocky. The primary joint set is oriented N40W, dipping 70-90NE with secondary jointing ranging from N05E, dipping 75SE to N60E, dipping 75SE.

DRILLING SUMMARY

7-69 Exploratory rotary and core drilling was conducted at the inlet portal, outlet portal and energy dissipator sites. One bucket auger hole was also drilled in the area of the outlet portal. No drilling was performed along the tunnel alignment.

Core and Rotary-Wash Drilling

7-70 Logs of holes drilled at the inlet portal, outlet portal, and energy dissipator sites are presented on plates A-21 to A-23 respectively. A summary of the rotary and core drilling conducted at these three areas is presented in tables A-37, 38 and 39.

Table A-37. Inlet Portal, Drilling Summary.

Hole No.	Rocktype	Bedrock Depth (ft)	Recovery (%)	RQD (%)	Discon. Freq. (per ft)	Point Load (lb/in^2)	K (ft/day)
R86-06	Diorite?	70.0	-	-	-	-	-
C86-07	Diorite	36.5	97	38	3.1	339	-
C86-30	Diorite	74.5	98	46	5.4	176	0.03
R86-31	Diorite?	87.5	-	-	-	-	-
Average:			98	42	4.3	258	0.03

Table A-38. Outlet Portal, Drilling Summary.

Hole No.	Rocktype	Bedrock Depth (ft)	Recovery (%)	RQD (%)	Discon. Freq. (per ft)	Point Load (lb/in^2)	K (ft/day)
C86-12	Diorite	67.5	88	28	3.9	265	-
C86-13	Diorite	60.6	97	38	3.3	60	-
C86-24*	Diorite	33.5	98	36	5.1	300	0.01
C86-25	Diorite	41.6	100	32	4.4	266	-
* inclined hole.							
Average:		56.6	96	34	4.2	223	0.01

Table A-39. Energy Dissipator, Drilling Summary.

Hole No.	Rocktype	Bedrock Depth (ft)	Recovery (%)	RQD (%)	Discon. Freq. (per ft)	Point Load (lb/in^2)	K (ft/day)
R86-08	-	68.5+			(bedrock not encountered)		
C86-10	Gneiss?	121.5?	16	0	10.0	-	-

Bucket Auger Drilling

7-71 One bucket auger hole, LAH-5, was drilled near the rotary and core holes at the outlet portal. The hole was not advanced beyond a depth of 19 feet because of severe caving in the dry colluvium; bedrock was not encountered. The soils log for this hole is shown on plate A-53 with the logs for bucket auger holes on the left abutment.

GEOPHYSICAL SURVEY RESULTS

7-72 The results of geophysical surveys conducted at three areas associated with the outlet works are presented below; no surveys were conducted for the tunnel alignment.

Refractive Seismic

7-73 Refractive seismic surveys were conducted at the inlet portal, outlet portal,and energy dissipator sites, see plate A-10. The time-distance graphs and subsurface interpretations are presented on plates A-36 through A-39.

Inlet Portal.

At least four velocity zones were identified at the inlet portal area. The velocity range for layer 1 (V1) represents dry, unconsolidated alluvium, layer 2 (V2) may represent more dense, unsaturated alluvium, layer 3 (V3) probably represents saturated alluvium and layer 4 (V4) represents bedrock. Correlation of refractive seismic data and drilling data from hole C86-30 is not good and the subsurface velocity model may be more complex. The measured velocities (V) and the approximate depths (D) are presented in table A-40.

Table A-40. Outlet Works - Inlet Portal, Results of Refractive Seismic Surveys.

Line No.	V1 (ft/s)	D1 (ft)	V2 (ft/s)	D2 (ft)	V3 (ft/s)	D3 (ft)	V4 (ft/s)
87-03	500-1,800	10	4,000	40	6,650	75	12,000
87-05	650-1,800	15	4,400	35	7,650	-	-

Outlet Portal.

Both refractive seismic lines at the outlet portal were conducted on the colluvium in order to roughly determine the overburden thickness. The velocity distinction between the unsaturated colluvium (V1) and the bedrock (V2) is quite distinct and the depth to bedrock (D1) correlates well with drill hole data. The measured velocities (V) and the depth to bedrock (D1) are presented in table A-41.

Table A-41. Outlet Works - Outlet Portal, Results of Refractive Seismic Surveys.

Line No.	V1 (ft/s)	D1 (ft)	V2 (ft/s)
86-03	675-2,000	65	12,000
86-08	700-2,100	55	9,500

Energy Dissipator.

The refractive seismic line conducted in the energy dissipator area yielded a velocity profile very similar to other lines conducted in the streambed. Three separate velocity zones corresponding to unsaturated alluvium (V1), saturated alluvium (V2), and bedrock (V3) are identifiable. The measured velocities and approximate interface depths are presented in table A-42.

Table A-42. Outlet Works - Energy Dissipator, Results of Refractive Seismic Survey.

Line No.	V1 (ft/s)	D1 (ft)	V2 (ft/s)	D2 (ft)	V3 (ft/s)
84-1/2	2,000-3,000	20	8,000	100	12,000

Downhole Seismic

7-74 Downhole seismic surveys were not conducted in holes associated with the outlet works.

LABORATORY TEST RESULTS

7-75 The results of laboratory tests conducted on core samples from the outlet works holes are presented in the following paragraphs.

Unconfined Compressive Strength

7-76 Unconfined compression tests were conducted on 2 core samples from the inlet portal area. The results of those tests are presented in table A-43.

Table A-43. Outlet Works - Inlet Portal, Results of Compressive Strength Tests.

Hole No.	Rocktype	Depth (ft)	Unconfined Compressive Strength (lb/in^2)	Point Load Index (lb/in^2)	Ratio
C86-07	Diorite	84.9	7,884	-	-
C86-30	Diorite	115.2	9,889	315	31.4

Rock Quality

7-77 The sample from hole C86-07 was also tested for specific gravity and absorption with the results shown in table A-44.

Table A-44. Outlet Works - Inlet Portal, Results of Rock Quality Tests.

Hole No.	Rocktype	Depth (ft)	Spec. Grav. (bulk SSD)	Absorption (%)
C86-07	Diorite	84.9-85.3	2.73	0.87

Petrographic Analysis

7-78 Petrographic analyses were not conducted on any samples from the outlet works.

Reservoir Area

MAPPING

7-79 Three distinct rock terranes are found in the reservoir. Upstream from the North Branch of the San Andreas fault zone, the reservoir rock is predominantly quartz monzonite (qm). The quartz monzonite is slightly to moderately weathered, forming steep to vertical faces above the streambed. The rock has pervasive quartzo-feldspathic dike intrusions with many along the two prominent joint sets of N40W, 60SW and N50-60E, 50SE. Along the North Branch fault zone, a sliver of the sedimentary Mill Creek formation (Tmc) is encountered in Warm Springs Canyon and on Government Canyon Ridge. South of the fault zone, the reservoir rock is the same gneiss and diorite found at the damsite. In addition to the major rock types mentioned, landslide debris, primarily rockfalls and relatively minor debris slides, is present on some of the lower slopes. An extensive area of stream terrace deposits mantled with colluvium on Government Canyon Ridge also would be inundated by the Seven Oaks Dam reservoir.

Downstream Pervious Borrow Area

FIELD TESTS AND RESULTS

7-80 Field tests for determining the characteristics of the downstream pervious borrow area were developed specifically to cope with the large size and percentage of rocks present in the borrow area. These tests are described in the following paragraphs.

Mass Gradation

7-81 In order to accurately estimate the amount of oversize materials in the Santa Ana River Wash, large samples ranging from approximately 24,000 to 58,000 lb were taken from each test pit and a gradation on the entire sample was determined. A series of hand held, single opening

screens was used to sort the 3-inch and larger stones, which were weighed in the field to determine the total weight of material passing each screen size. Stones too heavy to handle by hand were hoisted onto scales with a chain attached to a backhoe bucket. Boulders larger than the capacity of the scales, over 1,000 lb approximately, were measured in three directions and weights estimated by correlating with dimension-weight relations developed for smaller boulders that could be weighed.

7-82 The results of the field testing are shown on plate A-57 as gradation curves for each test pit. They show that the amount of the materials larger than 3 inches ranged from 49 to 59 percent of the total sample. The maximum size of the rock in the samples ranged from 20 to 36 inches in diameter, however, rocks up to 60 inches in diameter were excavated from the test pits and rocks up to at least 72 inches in diameter are visible within the proposed borrow area. In order to complete the mass gradation, five or six 600 lb samples of minus 3-inch material from each pit were taken for laboratory gradation testing.

Oversize Relations

7-83 Correlations between dimensions and weight were developed for boulders which are too heavy to be weighed under normal field inspection operations. By lifting with a chain sling and a backhoe, boulders up to 1,000 lb were weighed in the field. These weights were plotted versus the average of three girths that were measured in the field. The correlation between the weight and the average of the maximum and minimum girths, indicate that only the largest and smallest girths need to be measured in order to accurately estimate the weight of large stones from this borrow area. An alternate method of estimating the weight of large stones was to compute the weights based on a calculated volume and an assumed specific gravity. The volume of a sphere with a circumference equal to the average of the three girths measured in the field was used since the boulders tend to be subrounded in shape.

LABORATORY TESTS AND RESULTS

7-84 Approximately 35,000 lb of minus 3-inch material from TP86-3 were delivered to the South Pacific Division Soils Laboratory in Sausalito for detailed laboratory tests. The remaining materials were retained at the Los Angeles District project laboratory for gradation and maximum density testing.

Mechanical Analysis

7-85 Mechanical analysis tests were conducted on all of the minus 3-inch materials taken from the test pits. A mass gradation curve for each test pit, combining the plus 3-inch gradations determined in the field and the minus 3-inch gradations determined in the laboratory, is shown on plate A-57. Also shown on plate A-57 are the gradation curves for materials that would be produced if plus 12-inch and plus 3-inch rock were removed to provide upstream pervious shell and filter materials, respectively, for embankment fill.

Specific Gravity

7-86 Specific gravity tests were conducted on plus No. 4 and minus No. 4 size materials from TP86-3. Results, indicate a specific gravity of 2.66 for the plus No. 4 particles and a range of 2.59 to 2.64 for the minus No. 4 fraction.

Maximum Density

7-87 The maximum densities of the minus 6-inch and minus 3-inch portions of the samples were determined using vibratory compaction methods generally as described for the foundation alluvium. To insure that a true maximum density was achieved, the tests were run with increasing amounts of water until excess moisture bled out of the mold and no higher density was obtained. Extended periods of vibration were utilized on some tests to check whether complete settlement had occured during the standard test period. Minus 6-inch materials were tested in the 27-inch diameter mold and minus 3-inch materials were tested in the 27-inch and 11-inch diameter molds. All tests in the 27-inch mold were conducted with water added. Tests in the 11-inch mold were run both wet and dry, with the dry method generally producing a higher maximum density.

7-88 The results of these tests are shown graphically on plate A-58. They indicate that the maximum laboratory densities achieved in the 27-inch mold ranged from 137 to 146 lb/ft^3 for the minus 6-inch portions of the samples, and from 134 to 141 lb/ft^3 for the minus 3-inch portions. When using the 11-inch mold on the same minus 3-inch samples, the indicated maximum densities ranged from 126 to 135 lb/ft^3. In order to correlate the tests on different particle size ranges, a rock correction formula was applied to the minus 3-inch samples and the maximum densities adjusted to equivalent minus 6-inch material. The resulting maximum densities are shown in tabular form in table A-45.

Table A-45. Laboratory Maximum Densities of Downstream Pervious Borrow Area Materials.

Sample	Maximum Particle Size (inches)	Mold Size (inches)	Percent Oversize (lb/ft^3)	Lab Maximum Density (lb/ft^3)	Adjusted Maximum Density (lb/ft^3)
TP86-1	6	27	0	137.2	137.2
	3	27	30.4	129.3	136.1
	3	11	30.4	131.1	140.0
TP86-2	6	27	0	138.4	138.4
	3	27	34.3	132.8	139.7
	3	11	34.3	130.8	141.1

Table A-45. (Continued)

Sample	Maximum Particle Size (inches)	Mold Size (inches)	Percent Oversize	Lab Maximum Density (lb/ft^3)	Adjusted Maximum Density (lb/ft^3)
TP86-3	6	27	0	138.8	138.8
	3	27	25.0	132.3	137.7
	3	11	25.0	128.7	136.4
TP86-4	6	27	0	146.3	146.3
	3	27	34.4	134.7	141.3
	3	11	34.4	135.1	144.3
TP86-5	6	27	0	134.6	134.6
	3	27	27.9	126.9	133.6
	3	11	27.9	126.2	135.2

Triaxial Compression

7-89 Consolidated drained and consolidated undrained triaxial compression tests were conducted on 12-inch diameter samples from TP86-3 compacted to 95 percent of maximum density at 3 percent over optimum moisture content. The tests were run at confining pressures of 1, 2.5, 5 and 10 ton/ft^2, and the pore pressures were monitored during the consolidated undrained tests. The maximum size particle in these tests was 2 inches. The results are shown on plate A-59 in the form of p'-q' diagrams.

Permeability

7-90 Permeability tests were conducted on 12-inch diameter samples of materials from TP86-3 compacted at optimum moisture content to 90, 95 and 100 percent of maximum density. The results are shown on plate A-59. They show a range of permeability from approximately 23 to 315 ft/day.

Upstream Pervious Borrow Area

FIELD TESTS AND RESULTS

7-91 Test trenches 87-3 and 4 were excavated in the streambed upstream of the proposed embankment toe trace as part of the concrete aggregate investigation. They are within the area designated as the upstream pervious borrow area, and the gradations determined during the investigation are representative of the materials that would be

available from the site if it were used as a borrow source of zone 2, 3 and 5 embankment fill materials. The alluvial materials exposed during the investigation were not segregated into distinct layers of different gradations, except for isolated pockets of limited extent. A sample from each trench, therefore, was taken from the full depth of the trench, and is representative of all the materials that were encountered. A gradation of the plus 3-inch material from each of the trenches was conducted at the trench site in a manner similar to that described for the downstream pervious borrow area. The results are shown in the form of gradation curves for each trench on plate A-98.

LABORATORY TESTS AND RESULTS

7-92 A sample of the minus 3-inch portion of the materials from each test trench was taken for gradation and detailed laboratory tests for evaluation of the materials as concrete aggregate. For details of the test results, see section XI of this appendix. The tests applicable to use of the materials as embankment fill materials are described below.

Mechanical Analysis

7-93 A mechanical analysis of the minus 3-inch portion of each sample was conducted at the Los Angeles District soils laboratory. The results are shown on plate A-98.

Specific Gravity

7-94 The specific gravity of the materials from the trenches was determined for particles smaller and larger than 3/4 inches. The specific gravities ranged from 2.58 to 2.63 for the minus 3/4-inch materials, and from 2.63 to 2.66 for the plus 3/4-inch materials.

Impervious Borrow Area

FIELD TESTS AND RESULTS

Sand Cone Density Tests

7-95 Sand cone density tests were conducted at various depths in the top 15 feet of the soils in the preferred area of the borrow area shown on plate A-60 in order to determine the shrinkage factor to be expected when the materials are excavated and recompacted. A total of 34 tests were conducted on benches cut into the test trenches at depths ranging from 2 to 18 feet. Additional material was obtained adjacent to the density tests in order to investigate moisture-density relationships and determine the percent compaction of the in situ soils. ASTM D 698 standard compaction tests are expected to be the basis for quality control testing of the core and were used to determine the percent compaction of the in situ materials. The results of the in situ density tests indicate that the in situ dry density of the materials tested ranged from 88 to 127 lb/ft^3, with an average density of 114 lb/ft^3.

This corresponds to percent compactions ranging from 71 to 103 percent, with an average value of 93. The depth and density results for each in situ density sample are shown graphically on plate A-74.

LABORATORY TESTS AND RESULTS

7-96 Detailed tests to determine engineering properties were conducted at the South Pacific Division Soils Laboratory. Classification tests, a bentonite blending study and a portion of the compaction tests were conducted at the Los Angeles District project laboratory.

Mechanical Analysis

7-97 Grain size analysis tests were conducted on the minus 3-inch portion of all samples from the proposed borrow area. The results are shown on the soil logs on plates A-62 through A-73 and summarized on plate A-78 in the form of gradation curves. Although some areas within the indicated borrow limits have materials unsuitable for use in the impervious core, most of those within the areal and depth limits under consideration have between 10 and 50 percent fines and less than 15 percent gravel.

Atterberg Limits

7-98 Atterberg limits were determined for all samples that were not obviously nonplastic because of their granular nature. The results are shown on the soil logs. Generally, the core materials under consideration are not highly plastic. Liquid limits are generally between 20 and 40 percent, and the majority of the plasticity indices are less than 10 percent.

Bentonite Blending

7-99 Selected samples of potential core materials were blended with varying amounts of bentonite to evaluate the effect of the bentonite on some of the engineering properties that are considered crucial to acceptable performance in the embankment. These studies were initiated because of concern about the ability of the available core materials to deform sufficiently during earthquake induced motions, and to estimate the cost of alternative treatments if necessary. Three groups of samples were selected to represent the most plastic materials occuring in the borrow area (designated group SC), the least plastic materials available (group SM), and an area where borrow operations would be easiest and most economical to conduct (group SM-1). Each group of samples, approximately 600 pounds total weight each, was thoroughly blended and quartered before testing. One quarter of each sample was untreated and the other three quarter samples were blended with 1, 3 and 5 percent of dry powdered Wyoming bentonite. Each quarter sample was then subjected to mechanical analysis, Atterberg limits and moisture-density relationship tests to determine the effect of the varying amounts of bentonite on these core materials. Both standard (ASTM D 698) and modified (ASTM D 1557) compactions tests were conducted on each blended and untreated sample.

7-100 The results of the blending study are illustrated on plate A-79. The most obviously plastic and cemented materials that were visually identified (group SC) had an average of 36 percent fines and average liquid limit and plasticity index of 32 and 16 percent, respectively. The addition of 1 percent dry powdered bentonite had no significant effect. The addition of 3 percent bentonite raised the liquid limit of the blended sample to 39 percent and the plasticity index to 21 percent. Five percent bentonite increased these values to 47 and 28 percent, respectively.

7-101 The blend of visually determined low plasticity materials (group SM) had 35 percent fines and liquid limit and plasticity index of 23 and 4 percent, respectively. Again, the addition of 1 percent bentonite had very little effect on the Atterberg limits, raising each only 2 percent. Three percent bentonite raised the liquid limit to 29 percent and the plasticity index to 11 percent, and 5 percent brought the values up to 32 and 15 percent, respectively.

7-102 The blend of samples from the easily accessible area (group SM-1) had an average of 32 percent fines and liquid limit and plasticity of 23 and 6 percent, respectively. The addition of 1 percent bentonite increased the liquid limit to 27 percent and the plasticity index to 10 percent. The addition of 3 and 5 percents of bentonite produced values of 30 and 13 and 33 and 15 percents, respectively for the liquid limit and plasticity index.

7-103 For all of the samples, the general trend of the compaction curves when increasing amounts of bentonite were added was to lower the maximum density and increase the optimum moisture content. Based on the results of this preliminary study, selected samples were blended with 3 percent bentonite and subjected to the full range of detailed tests described for core materials. The results of these tests are shown on plate A-76.

Moisture-Density Relationships

7-104 Standard effort and modified effort compaction tests, corresponding to ASTM D 698 and 1557 test methods, were used to determine the moisture-density relationships of the potential core materials. These results, shown on plate A-78 in the form of 5-point compaction curves, were used to prepare samples for subsequent testing and to estimate shrinkage factors for borrow operations. The range of maximum densities for impervious core materials was between 114.7 and 125.0 lb/ft^3 for ASTM D 698 methods of compaction, and between 125.4 and 133.4 lb/ft^3 for ASTM D 1557 compaction procedures. Generally, the results were consistent for all tests, with ASTM D 698 procedures producing lower maximum densities, higher optimum moisture contents, and somewhat flatter curves.

Consolidation

7-105 Consolidation tests were conducted on untreated and bentonite blended samples of core materials compacted to 98 percent of maximum density (ASTM D 698) at optimum moisture content and at plus and minus 2 percent of optimum moisture content. These values were chosen as being representative of probable placement conditions during embankment construction. The results, shown on plate A-75, were used in the settlement analysis of the embankment.

Permeability

7-106 Permeability values were determined for core materials compacted to 90, 95 and 100 percent of maximum density (ASTM D 698) at optimum moisture content. Both untreated and bentonite blended samples were tested. The results are shown on plate A-75. They indicate that the expected range of permeability of the available core materials when placed and compacted in the embankment would be between 0.0028 and 0.1 ft/day. If bentonite is added, the range would be expected to be between 0.00076 and 0.026 ft/day.

Triaxial Compression

7-107 Consolidated undrained (R) triaxial compression tests were conducted on both untreated and bentonite blended samples, with pore pressures monitored to obtain consolidated drained (S) strengths also. The samples were compacted to 98 percent of maximum density (ASTM D 698) at 2 percent over optimum moisture content, and tested at confining pressures of 1, 2.5, 5, 10, 15 and 20 ton/ft^2 to represent various depths of overburden in the embankment. The results are shown on plate A-77.

Dispersion

7-108 SCS type dispersion tests were conducted on both blended and untreated samples to determine if dispersive clays are present in the proposed core borrow materials. Pinhole dispersion tests were found to be unsuitable for these materials because the holes would not stay open. The results of the tests indicated that the untreated materials are nondispersive, but that some of the core materials treated with Wyoming bentonite may have a tendency to disperse when saturated.

Specific Gravity

7-109 Specific gravity tests were conducted on selected samples of core material. The plus and minus No. 4 portions were determined separately. Results indicate a range of 2.67 to 2.73 for the plus No. 4 materials, and from 2.68 to 2.73 for the finer fraction.

North Branch Fault Investigations

7-110 The fault investigations along the North Branch fault zone yielded valuable information on the nature of faulting and recency of activity. The primary conclusion of the studies conducted thus far is that faulting associated with the North Branch zone offsets a late Pleistocene terrace which is estimated to be about 55,000 years old. In trench T87-2, a six to twelve inch wide gouge zone juxtaposes quartz monzonite (qm) on the northeast with the Mill Creek formation (Tmc) to the southwest. Two lithologic units of the formation were observed. Immediately west of the fault is a ten-foot wide zone of dark-brown mudstone, in sharp contact further west with a reddish conglomerate consisting of cobble to boulder-sized clasts in a sand-silt matrix. Detailed inspection of the colluvium overlying the fault in the bedrock, indicated slight offsets. Re-excavation and inspection of the colluvium in trench T87-1 indeed confirmed that the overburden materials in an area about 200 feet long have been displaced on the order of inches to several feet. See plate A-48 for a detailed log of trench T87-1. Based upon these limited exposures, the fault zone is characterized by a graben-like structure with apparent offsets of several feet. Preliminary assessment is that the features observed could be the result of dip-slip movement with the north-side down or a small pull-apart associated with lateral offsets. The amount of lateral offset is unknown, however, reasonably good matches in the colluvial facies suggest only a few tens of feet at most.

7-111 Previously published geologic maps of the area had shown the Mill Creek formation sediments to pinch-out in Santa Ana River canyon. They also had shown the North Branch to be composed of a single trace referred to as the Mill Creek fault (Allen, 1957; Matti, Morton and Cox, 1985). Based upon this new evidence, it appears that the North Branch fault zone is composed of at least two strands which bound the sedimentary Mill Creek formation. Work will continue during the FDM study phase to (1) expose the contact on the south side of the zone, (2) better establish the amount of lateral movement in the zone, and (3) more accurately determine the age of terraces and colluvium offset by the fault(s).

7-112 In spite of this new evidence, it is still believed that the South Branch is the modern neotectonic strand of the San Andreas fault in the vicinity of the damsite. Over the last several tens of thousands of years, the slip-rate has been at least one and possibly two orders of magnitude more than that on the North Branch zone (Weldon, written comm.). Therefore, a magnitude 8+ earthquake on the South Branch at the mouth of the canyon is still considered to be the design earthquake event for the Seven Oaks Dam and no additional design measures are necessary related to the North Branch.

VIII. ANALYSIS OF DATA

Streambed Foundation Bedrock

DEPTH AND CONFIGURATION OF BEDROCK SURFACE

8-01 Based upon the depth to bedrock as determined in the 21 core holes, 7 rotary holes and 9 Becker Hammer holes drilled in the streambottom under the central portion of the proposed dam, a top of bedrock contour map, plate A-41, was prepared. Refractive seismic information was of very little use in the preparation of the map because of: (1) the irregularity of the bedrock surface, (2) the narrowness of the canyon bottom relative to the bedrock depth, and (3) the indistinct velocity differential between the coarse, saturated alluvium and the foundation bedrock. As plate A-41 indicates, the Santa Ana River has apparently eroded into the gneiss and diorite bedrock an assymetrical and well defined channel which approximates the present canyon configuration. The deepest portion of this channel at the damsite is approximately 100 feet below the existing ground surface. The variability of the bedrock depth in relatively close-spaced holes such as C84-01 and C86-29 (57 feet), C86-33 and R86-41 (62 feet), and C86-35 and C86-38 (51 feet) indicates steep relief and a very irregular bedrock surface. An apparent shallow bedrock shelf at the left abutment may actually contain a secondary incised channel. This can only be determined by additional drilling which will be conducted during the FDM study and ultimately by actual streambed alluvial excavation. Bedrock contours outside of the area of concentrated exploration are purely conjectural.

BEDROCK CHARACTERISTICS

8-02 The foundation under the central portion of the dam will be both gneiss and diorite. The contact between the two appears to be irregular and gradational. The physical properties of the two rockmasses are almost identical as shown by the comparison shown in table A-46.

Table A-46. Overall Drilling Summary, Streambed Foundation Bedrock.

Item	Recovery (%)	RQD (%)	Discontinuity Frequency (per foot)	Point Load Index (lb/in^2)	K (ft/day)
All drilling	85	25	6.2	151	0.06
Diorite	83	25	6.7	155	0.06
Gneiss	86	25	5.8	147	0.06

Likewise, the average degree of weathering is very similar with the gneiss generally moderately to highly weathered and the diorite moderately weathered. Generally, the overall quality of the bedrock does not markedly improve with depth as shown on plates A-30 through A-35. However, where a distinction can be made, the depth to improved rock quality varies from approximately 10 to 30 feet below the bedrock surface.

8-03 In summary, the streambed foundation bedrock under the proposed dam is a moderately to highly weathered, highly fractured gneiss and diorite with rockmass permeability less than 0.1 ft/day. Although the inclination of discontinuities is quite variable, those between 60 and 80 degrees seem to predominate. Almost one half of the discontinuity surfaces show indications of movement such as slickensides or clay gouge seams up to 1/8-inch thick, however, compressible clay/gouge zones of significant thickness are not in evidence. Rock strength, as determined from unconfined compression tests on intact core samples, is approximately 5,000 lb/in^2. An average strength value of 4,000 lb/in^2 for the overall rockmass is considered reasonable. The rockmass quality does not improve enough with depth to warrant over-excavation of the bedrock foundation surface.

FAULTING

8-04 It is anticipated that, when the foundation bedrock is fully exposed and cleaned, a large number of bedrock shears will be encountered. However, based upon detailed geologic mapping of bedrock adjacent to the streambed foundation area, it is unlikely that major fault zones will be uncovered. The general rockmass and topographic continuity, both laterally and vertically, would indicate that no major movements have occurred at the damsite. Neither offsets in the overburden nor in the overlying terrace deposits, which might indicate capable faulting at the damsite, have been observed during geologic mapping of the area or inspection of the numerous access road cuts at the left abutment.

Streambed Foundation Alluvium

8-05 For anticipated conditions during operation of the dam, the upstream portion of the alluvial foundation is assumed to be completely saturated from impoundment of a debris pool. Outlet releases near the downstream toe combined with a groundwater barrier related to the South Branch of the San Andreas fault approximately 4,000 feet downstream will saturate much of the downstream portion also. For this condition it is apparent that the density, gradation and liquefaction potential of the foundation materials have a major impact in the embankment stability assessment. Based on the investigations and testing to date, this report will demonstrate that the foundation materials below a depth of about 20 feet appear to be relatively uniform, adequately dense and pervious enough to minimize pore pressure build up, thus retaining most of their static undrained shear strength so that they would safely support the proposed dam under strong earthquake motions that may develop at the site. In the subsequent FDM study phase, additional field testing will be performed to verify the in situ relative density, permeability and gradation to show that they are sufficient to nullify the concern about potential liquefaction.

GRADATION

8-06 Within the context of the general alluvial environment of the area, as indicated by trench and drill logs, the foundation materials do not vary significantly over the extent of the embankment foundation area. Gradation tests on materials from trenches excavated at the Mentone and Lytle Creek damsites nearby are similar in terms of gradation and stratification. No conditions were encountered that would indicate anything other than consistency of foundation conditions. These investigations indicate that materials deposited under intermittent, high-velocity streamflows are generally very coarse and reasonably consistent from one area to another. It is assumed therefore that, throughout the foundation area, the alluvial materials consist of cohesionless, coarse-grained gravelly sands and sandy gravels with occasional sand lenses of limited extent and relatively large amounts of cobbles and boulders up to at least 7 feet in diameter. Based upon mass gradations conducted in the streambed, it is estimated that 40 to 70 percent of the material may be larger than 3 inches and less than 20 percent smaller than Number 4 sieve size. The oversize rock is generally subrounded to subangular, with relatively few fresh fractured faces.

DENSITY

8-07 The limited in situ density data obtained from tests in the three test trenches, in conjunction with geophysical seismic survey data, indicate that the foundation materials become denser with depth. Although some of the density test results are questionable, the others are in reasonably good agreement with density test results on similar materials at the nearby Mentone and Lytle Creek damsites. Based on these density tests, seismic survey results, drilling action, and

observation of materials in deep trenches downstream, it is reasonable to assume that the average densities would be at least 135 lb/ft^3 within the embankment foundation below depths of 20 feet. To assure a dense embankment foundation, the top 20 feet of alluvium would be removed prior to construction of the embankment, and all streambed materials would be removed in the central portion of the embankment, as described in paragraph 10-12.

8-08 Although the in situ density data indicates that the foundation materials are at an average relative compaction of about 94 percent, these tests were performed in the top 20 feet where the materials are known to have lower density. The average in situ density at the bottoms of the trenches was 136 lb/ft^3. Information from Los Angeles District construction projects having materials with similar gradations indicates that these coarse-grained materials can be compacted to an average density of about 138 lb/ft^3. Therefore, the materials below about 20 feet are probably as dense as present construction methods can compact them. The results of the aquifer test conducted in the foundation indicate a lower permeability and a much lower storativity than would be expected from analysis of effective grain sizes. Since the storativity of an aquifer is a function of the size and number of interconnected voids, it may reasonably be judged that the low values are evidence of either low void ratios in the materials below the water table and therefore, high densities, or horizontal layers of fine and coarse material. Explorations to date do not indicate the presence of extensive horizontal layers of either fine sands or open gravels. These results, along with the construction control records from other projects using similar materials, verify the field and laboratory test results and support the conclusion that alluvial outwash deposits of this type are naturally dense.

LIQUEFACTION POTENTIAL

8-09 A review of the data on gradation and density of the foundation materials at the Seven Oaks damsite indicates that the materials are coarser and denser than alluvial deposits of the kind that literature indicates in the past have shown evidence of liquefaction. Even if there were a tendency for excess pore-pressure development, there should be less concern about significant strength loss and deformation because of the following considerations:

a. The foundation materials are coarse-grained cohesionless soils with relatively high permeabilities. Calculated permeabilities from an aquifer pump test conducted in November 1984 ranged from 240 to 340 ft/day. Laboratory permeability tests on the minus 3-inch portion of similar materials compacted to densities expected in the foundation ranged from about 23 to 315 ft/day. Because of the relative coarseness and relatively high permeabilities, the foundation materials would be subjected to partial drainage under seismic loading. This partial drainage would increase the dynamic strength of the soil over the undrained dynamic strength. The presence of extensive layers of

fine-grained uniform sand in the foundation would be a matter of concern, since drainage of groundwater in the sand layer might not be sufficient to reduce pore pressures within that layer generated in a seismic event. However, nothing other than relatively limited sand pockets which could easily drain to adjacent coarse materials have been encountered to date in exploration trenches. Localized high pore pressures may develop temporarily in sandy deposits with minor loss of strength, however drainage to adjacent materials is expected to be rapid with no adverse effects to the foundation.

b. Coarse-grained, relatively dense cohesionless soils tend to develop less strains than the kind of alluvial deposits that have historically shown evidence of liquefaction (Wong, 1971; Banerjee, Seed and Chan, 1979). The foundation materials at Seven Oaks damsite, under dynamic loading, may at the very worst have the potential to develop a condition of initial liquefaction with limited strains. "Initial liquefaction with limited strain potential," as described by Seed et. al. 1975, "denotes a condition in which cyclic stress applications develop a condition of initial liquefaction and subsequent cyclic stress applications cause limited strains to develop either because of the remaining resistance of the soil to deformation or because the soil dilates, the pore pressure drops, and the soil stabilizes under the applied loads." Tests conducted on Oroville gravels (Wong et. al., 1974; Banerjee et. al., 1979) indicate limiting axial strains on samples at 60 to 84 percent relative density ranged from +2.5 to over +10 percent.

c. It is reasonable to assume that previous strain history from earthquakes dynamically loading the foundation materials at the site may have increased the material's resistance to liquefaction. Studies conducted by Lee and Focht (1975) indicate that previous strain histories led to an increase in cyclic strength for dense sand by a factor of at least 1.5.

8-10 Because of the previous three reasons, there are few known instances of liquefaction type phenomenon or large deformations recorded in gravelly soils. A study report by Wong et. al. (1974) on movements of bridge foundations during the Alaska earthquake of 1964 indicates large movements of foundations founded in sandy soils, with much smaller movements for foundations founded on gravelly soils. Historical data for stress ratios causing conditions of liquefaction and nonliquefaction in the field indicate that the gravels did not liquefy under loading conditions that caused liquefaction in sands.

8-11 In summary, historical and recent test data indicate that coarse-grained, cohesionless, relatively dense foundation materials of the type in the Santa Ana River streambed at the Seven Oaks damsite would not undergo damaging liquefaction. The coarse-grained materials would probably develop limiting dynamic strains on the order of +3 to +5 percent. To verify the foregoing assumptions, an extensive

investigation of the foundation materials and conditions will be conducted during the FDM study phase. See paragraphs 13-03 through 13-08 for a description of the foundation investigation portion of the test fill contract.

Right Abutment

OVERBURDEN PROPERTIES

8-12 The overburden on the right abutment is primarily a mixture of residual soil and slope wash with occasional remnants of alluvial terrace deposits. The terrace material is coarse, cohesionless sand, gravel, cobbles and boulders to several feet in diameter. The residual soil is loose silty sand which supports a grassy chapparal vegetation. Primarily because the colluvial deposits are thin and irregular in extent and somewhat heterogeneous in composition, they will not be utilized in the construction of the dam embankment. The overburden on the right abutment should be easily excavated.

BEDROCK PROPERTIES

8-13 Bedrock at the right abutment is almost equally diorite and gneiss with the gradational contact semi-parallel to the embankment centerline. Although almost no subsurface exploration has been conducted at the right abutment, it is believed that the rockmass properties of the diorite and gneiss will be found to be very similar to those of the streambed foundation bedrock. Therefore, the rock is expected to be highly fractured, moderately to highly weathered with low permeability. Field mapping indicates steeply dipping, generally east-west foliation within the gneiss. Locally, mafic rich zones within the gneiss may be more deeply weathered and therefore, require additional excavation or surface treatment.

FAULTING

8-14 Several bedrock faults were identified during the geologic mapping of the right abutment, however, they were either of limited extent or covered by unfaulted colluvium. General overall continuity of the rock units and topography indicate no large scale faulting through the abutment.

Left Abutment

OVERBURDEN PROPERTIES

8-15 The majority of the overburden materials on the left abutment are contained in a relatively deep deposit of colluvium along the centerline of the proposed embankment. Drill holes show that this deposit is over 100 feet thick in the lower portions and contains about 500,000 yd^3 of material. The soil logs of bucket auger drill holes in this deposit are shown on plate A-53. The logs show that the majority of the materials

below the surface are nonplastic sands and silty sands with varying amounts of gravel and cobble-size decomposed rock fragments. This deposit comprises approximately 80 percent of the overburden on the left abutment area. The remainder is similar to the right abutment overburden previously described, a relatively thin cover of topsoil and weathered rock fragments, mostly diorite. The surficial materials appear to be too variable and contain too much organic material to be suitable for embankment fill. They will be wasted or possibly used in landscape or miscellaneous fill areas if needed. The large mass of colluvial sands is relatively consistent throughout its depth and would be suitable for use as a road construction material or in the downstream transition portion of the embankment if it can be excavated without undue contamination with unsuitable organic surficial materials.

BEDROCK PROPERTIES

8-16 Only two core holes were drilled at the left abutment and only one of those under the central portion of the proposed embankment. However, based upon surface mapping at the abutment and the inspection of the Edison tunnel, it is believed that the rockmass at the left abutment is almost exclusively diorite. The properties of the diorite rockmass appear to be comparable to those determined for the diorite bedrock in the streambed foundation as shown in table A-47.

Table A-47. Streambed Foundation/Left Abutment, Drilling Summary Comparison.

Item	Recovery (%)	RQD (%)	Discontinuity Frequency (per foot)	Point Load Index (lb/in^2)	K (ft/day)
Diorite Foundation	85	25	6.2	151	0.06
Left Abutment	86	26	6.1	234	0.05

8-17 Based upon the limited explorations conducted thus far, the quality of the rockmass does not appear to significantly improve with depth. As with most of the other holes drilled through the diorite rockmass, the point load strength index values remain remarkably constant with increasing depth. Although the unconfined compressive strength of one sample from hole C84-06 was over 13,000 lb/in^2, three samples from C86-04 averaged only 3,794 lb/in^2. This value is reasonably in line with the assumed overall 4,000 lb/in^2 strength of the diorite. Rockmass permeability should be on the order of 0.1 ft/day. It appears that the discontinuities in the rockmass generally dip steeper than 45 degrees and, as with the streambed foundation rockmass,

the majority show evidence of differential movement. Thin clay-gouge seams and slickensided surfaces are abundant. When the left abutment bedrock is exposed during abutment excavation, it is expected that the surface will be moderately to highly weathered and highly jointed and sheared. It is estimated that 10 percent of the bedrock will require blasting during the abutment excavation access road construction.

FAULTING

8-18 It is anticipated that when the left abutment is fully exposed and cleaned during the FDM study phase, a large number of bedrock shears will be encountered. However, based upon detailed geologic mapping of bedrock outcrops on the left abutment, it is unlikely that major fault zones will be uncovered. No active or potentially active faults have been identified at the damsite and the general rockmass and topographic continuity both laterally and vertically indicate no major movements at the damsite.

Spillway

OVERBURDEN PROPERTIES

8-19 Thin overburden deposits effectively conceal the upstream half of the spillway area. The overburden is almost exclusively residual soil and slopewash less than 3 feet thick. No testing has been conducted on the material but visually it appears to be a silty sand which supports a thick chaparral vegetation. The material will be easily excavatable. Because of the small volume and the mixture of woody vegetation it will not be used in the construction of the embankment.

BEDROCK PROPERTIES

8-20 Because of the large volume of material from the spillway to be excavated and used in the embankment, a great percentage of exploration and testing conducted during the Phase II GDM investigations was aimed at determining the expected properties of the rockmass. The majority of the spillway will be excavated in gneissic rock with the remainder in diorite. The gneiss is more heterogeneous in composition and physical properties than the diorite. Nonetheless, the degree of overall variability is quite limited. As determined from the drilling summary (table A-24), the spillway rock is highly fractured, moderately weathered and only moderately hard. Downhole and refractive seismic surveys indicate P-wave velocities below 7,000 ft/s in the upper 100 feet. Rock quality comparisons on plates A-32 to A-34 indicate that a surficial zone of poorer quality rock is identifiable in at least 6 of the 8 core holes drilled. This surficial zone varies in thickness from 36 to 132 feet. The deeper rock, although relatively higher in quality, is still highly fractured and only moderately hard. Laboratory tests confirm that the rock quality, as measured by abrasion, absorption and wetting and drying results is not related to depth. The overall physical properties of the rockmass appear to be the direct result of compressional stresses and the degree of microfracturing.

Excavatability

8-21 Textural and compositional variation within the gneissic rockmass will likely cause variability in the excavatability of the spillway. However, due to the uniformly high degree of fracturing and relatively low strength of the rockmass in general, it is estimated that 90 percent of the spillway can be mechanically excavated.

Erodibility

8-22 A quantitative estimate of the erodibility of the spillway rock floor has not been attempted. It can be assumed that the physical properties of the rockmass which might make it rippable would also tend to make it more easily erodible (Cameron, et. al., 1986). Because the gneissic rockmass is highly fractured and variable in hardness, it is reasonable to expect that spillway flows of high velocity could produce localized erosion and plucking of small rock blocks. Nonetheless, given the remote probability of occurrence and the moderate velocities and short duration of flow associated with a spillway flood, concrete lining of the spillway is not recommended at Seven Oaks Dam.

Rock Quality/Size

8-23 The spillway rockmass in-situ is highly fractured with an average of 6 discontinuities per foot of recovered core and an average RQD of approximately 30 percent. Core losses in the representative holes averaged 9 percent. On the basis of these factors, the following gradation is estimated for rock excavated from the spillway after wasting the most highly weathered and degraded rock:

Table A-48. Estimated Spillway Rock Gradation.

Size	Percent Passing
12 inch	100
6 inch	80
3 inch	60
no. 4	30

However, laboratory tests conducted thus far indicate that the spillway rock may be susceptible to significant breakage during hauling, placement, and compaction. The actual excavated gradation and the eventual gradation after placement and compaction can be estimated with a greater degree of certainty only after the proposed test excavation in Government Canyon Ridge and the associated test fill during the FDM study phase.

Cut-Slope Stability

8-24 The cut-slope stability in the spillway excavation will ultimately be controlled by the geometry of the excavation and the discontinuities in the rockmass. The prevailing direction of the steeply dipping foliation in the gneiss is almost perpendicular to the proposed spillway alignment. Unfortunately no prevailing joint sets were identified during the field mapping. The inclination frequency plot of discontinuities measured in the 8 cores from the spillway shows a predominance between 40 and 75 degrees with 40 and 60 degrees being the most common. It is not possible to determine the strike of these discontinuities and therefore the effect they might have on slope stability. Since, however, over 50 percent of the logged discontinuities showed evidence of either slickensides or clay/gouge, where the overall orientation is adverse, localized slope stability problems are likely to occur.

FAULTING

8-25 As shown on plate A-6, two faults were mapped in the spillway which offset a mafic-rich zone in the gneiss. Since they appear to be very steeply dipping and project mainly to the floor of the spillway excavation, they should not pose special slope stability problems. They may, however, intersect the concrete control sill. An attempt will be made to more accurately determine the relationship of these faults to the sill during the FDM studies. The final assessment of their significance on the stability of the sill and possible higher erosion potential will be made during construction, when the spillway is fully excavated.

Government Canyon Drainage Channel

OVERBURDEN PROPERTIES

8-26 The west side of the proposed drainage channel excavation, and Government Canyon Ridge in general, is covered by thin residual soil and thick woody vegetation. The overburden material will be wasted along with the stripped vegetation.

BEDROCK PROPERTIES

8-27 All of the bedrock to be excavated from Government Canyon Ridge will be from the gneissic rockmass. Although there is variability in both the compositional and physical properties which precludes extrapolation between core holes, the overall characteristics of the rockmass are quite similar. As determined from the drilling summary (table A-18), the Government Canyon Ridge rock is highly fractured, moderately to highly weathered and only moderately hard. Downhole and refractive seismic surveys indicate P-wave velocities less than 3,500 ft/s in the upper 50 feet and less than 8,000 ft/s to depths as much as 180 feet. Rock quality comparisons on plates A-32 and A-34

indicate that a surficial zone of poorer quality rock is identifiable in each of the 4 core holes drilled in the Ridge. This surficial zone varies from 34 to 84 feet in thickness. Laboratory tests confirm that the rock quality, as measured by abrasion, absorption and wetting and drying results is not related to depth. The overall physical properties of the rockmass appear to be the direct result of compressional stresses and the degree of microfracturing.

Excavatability

8-28 Textural and compositional variation within the gneissic rockmass will likely cause variability in the excavatability of Government Canyon Ridge. However, due to the uniform high degree of fracturing and relatively low strength of the rockmass in general, it is estimated that 90 percent of the rock can be mechanically excavated. The proposed test excavation in the area of the drainage channel excavation during the FDM study phase will yield qualitative and quantitative data on the effectiveness of different excavation techniques which will be used in formulating cost estimates and specifications for the embankment construction contract.

Rock Quality/Size

8-29 The Government Canyon Ridge rockmass in situ is highly fractured with an average of 5.6 discontinuities per foot of recovered core and an average RQD of 31 percent. Core losses in the four core holes averaged 14 percent. On the basis of these factors, the following gradation is estimated for rock excavated from Government Canyon Ridge after wasting the most highly weathered and degraded rock:

Table A-49. Estimated Government Canyon Ridge Rock Gradation.

Size	Percent Passing
12 inch	100
6 inch	80
3 inch	60
no. 4	30

However, laboratory tests conducted thus far indicate that the rock may be susceptible to breakage during hauling, placement, and compaction. The actual excavated gradation and the eventual gradation after placement and compaction can only be estimated with a greater degree of certainty after the proposed test excavation in Government Canyon Ridge and the associated test fill during the FDM study phase.

Cut-Slope Stability

8-30 The rockmass conditions in Government Canyon Ridge which affect slope stability are similar to those in the spillway. Discontinuities in the core from the four holes drilled on the ridge are inclined predominantly 35 to 70 degrees with those dipping 40 to 45 degrees being the most common. Like in the spillway area, over half show evidence of slickensides or clay/gouge. However, the orientation of the drainage channel excavation (N57W) more closely follows the trend of the steeply dipping mineral foliation (N80E to N70W).

Spillway/Government Canyon Ridge Rock Quality Comparison

8-31 A test excavation and test fill of the rock to be utilized in the embankment construction will be conducted to determine the physical properties of excavatability, gradation, strength and permeability. Ordinarily, a test excavation would be conducted in the spillway, where over 7 million yd^3 will be removed during construction of the embankment. However, on the basis of the drilling and laboratory test data as listed in table A-50, the rockmasses in the spillway and Government Canyon Ridge appear to be similar to each other and data from a test excavation in Government Canyon Ridge can be used for design purposes to represent rock excavated from both locations.

Table A-50. Spillway/Government Canyon Ridge, Rockmass Comparison.

Parameter	Government Canyon Ridge	Spillway
Recovery Percentage	86	91
RQD, percent	31	39
Discontinuity Frequency	5.6	6.3
Point Load Index, lb/in^2	177	162
Downhole P-wave Velocity, ft/s	3,330-8,000	2,080-6,490
LA Abrasion Loss, percent	81.1	78.5
Bulk SSD	2.63	2.55
Absorption, percent	2.29	2.81
Plagioclase/orthoclase	52/48	64/36
Quartz, percent	15	16
Mafics, percent	12	15

OVERBURDEN PROPERTIES

8-32 Overburden that will be impacted by the outlet works is limited to the inlet and outlet portals. At the inlet, overburden consists of mostly streambed alluvium capped with a small amount of colluvium from the slopes above. At the outlet, the portal is covered with colluvium as is the initial part of the outlet channel. The remainder of the outlet channel and the plunge pool will be founded in streambed alluvium. In general, the alluvium consists of clean, coarse sand with sub-rounded gravel, cobbles and boulders to several feet in diameter. The colluvium differs in being more of a silty sand with angular rock fragments reaching a maximum size of about one foot.

BEDROCK CONFIGURATION

8-33 Although configuration of the bedrock surface is not fully known at both portal areas, general trends have been determined for pre-FDM phase design of the outlet structures. See plate A-47 for geologic profile and subsurface bedrock contours. At the inlet, the ridge face immediately above the valley floor is a massive rock outcrop on an overall slope of about 1V on 1H. Below the valley alluvium which intersects the face at about elevation 2,140, the bedrock surface contours appear to be on a slight skew to the outlet works centerline, dropping short on the right side. At the proposed outlet, little is known of the depth to bedrock along the channel between the portal and the plunge pool. At the portal, the bedrock surface is only slightly steeper than the ground surface; daylighting about 200 feet upslope from the tunnel exit site. At the portal, the colluvium appears to be about 40 feet deep vertically, however, the bedrock surface appears to be irregular. Under the plunge pool, the bedrock surface lies between 85 and 120 feet below the ground surface. Further description of the bedrock configuration can be found in Volume 1, Appendix B, Seven Oaks Dam Outlet Works, by the Portland District.

BEDROCK PROPERTIES

8-34 The outlet works will be constructed mostly, if not entirely, in diorite. As exposed in the local outcrops and explored in core holes at the portals, the diorite is generally moderately hard to hard, medium grained, somewhat altered and strongly jointed; patterns, however, are random and joint sets change frequently. Joint spacing also varies, but averages between 1 and 2 feet making the rock "blocky". Although the exploration indicates the joints are tight, most are coated and little tensile strength exists between blocks. Upstream from the vicinity of the dam centerline to the inlet area, the diorite appears to be the most sound and competent of any at the project site. Downstream, the rock is possibly more fractured, sheared and weaker. Other description of the bedrock can be found in Volume 1, Appendix B.

Excavatability

8-35 The excavatability of the main outlet works features are discussed below.

Tunnel

8-36 It is anticipated that the tunnel will encounter rock of a similar condition as that found in the Edison tunnel farther up the ridge. The rock will be hard enough to require blasting throughout and sufficiently "blocky and seamy" to also require continuous support. However, it is anticipated that the upstream half will extend through more competent rock than the downstream half.

Portals

8-37 Blasting will be required to excavate the bedrock at both portals. The outlet portal, however, may be impacted by faulting which may require other methods for excavating broken rock. Further exploration is planned for the FDM study phase to determine the presence and extent of faulting. Some support will be required of all cut slopes in rock to minimize stress relief of the blocky rock. Overburden at both portals should be easily excavated by common equipment.

Channels

8-38 Most of the approach and outlet channels will be excavated through overburden; some bedrock likely will occur in the floors adjacent to the portals. The overburden should be easily excavated by common methods although an occasional large boulder may require blasting near the inlet and also in the vicinity of the plunge pool. Cut slopes should be no steeper than 1V on 2H.

Tunnel Rockmass Classification

8-39 Although no exploration has been conducted along the tunnel alignment, the bedrock is expected to be similar to that explored elsewhere in the project area and exposed in road cuts, outcrops, and the Edison tunnel. Based on preliminary data and as analzyed by the Portland District in Volume 1, Appendix B, two rock classifications have been determined. The Terzaghi classification is used to determine rock loads and the appropriate size and spacing of steel sets for tunnel support. For the tunnel sizes being evaluated, the rock is considered to be a class 5 (very blocky and seamy with little or no side pressure). The rock load is estimated to be 0.5 x (B+H) x Wr where B is the tunnel width, H is the tunnel height and Wr is the unit weight of the rock. The second classification is the RMR (Rock Mass Rating) system. Under this system, the rock is classified as being between a class IV (poor) or a class III (fair).

Slope Stability

8-40 Slope stability is a consideration of both portal cuts and the channel sideslopes. At the inlet, the approach channel sideslopes are mostly in alluvium which would be stable on slopes flatter than 1V on 2H until the first inundation, when they will form their own natural slopes, if left unprotected. At the inlet portal, temporary cuts in bedrock will be stable on near vertical slopes with support such as rock bolting. For the permanently exposed cuts above the portal, the slope will be 1V on 0.25H, also with support. The existing rock face is presently stable and maximum effort will be made to disturb the face as little as possible by minimizing the excavation. At the outlet, the colluvium overlying the portal will be removed probably at least down to the top of the channel. As at the inlet, cuts in rock will be near vertical for temporary and 1V on 0.25H for permanent slopes and supported. Should a fault intersect the cut and the rock be more broken than anticipated, the stability treatment will be adjusted by increasing the support and/or flattening the slopes. For the outlet channel, both rock and alluvial slopes will be temporary since the channel will be lined and the walls backfilled. Cuts in the rock will be short and can be 1V on 0.25H without support and probably 1V on 2H in the alluvium. A more detailed description is presented in Volume 1, Appendix B of this Phase II GDM.

FAULTING

8-41 Several narrow faults, exposed elsewhere on the left abutment, project towards the tunnel nearly perpendicular to it. Fault "A", which forms the downstream contact between the diorite and gneiss, projects nearby to the proposed outlet portal. Other faults likely exist upstream from Fault "A" as evidenced by several full concrete sections in the Edison tunnel. It is anticipated that such faults will not exceed a foot or two in width and will be steeply dipping. It is also anticipated that numerous hairline shears are present and that some of them may require special treatment.

8-42 One of the seismic considerations adopted for this project is the possibility of a maximum 4-foot displacement subsidiary to a great event on the San Andreas fault. The maximum displacement would likely occur on existing strong shears likely dissecting the site. The regional trend of shearing is generally perpendicular to the outlet alignment, therefore, the possiblity of up to a 4-foot offset somewhere in the tunnel should be considered. The preliminary field investigations indicate that candidate shears are most likely to occur in the downstream half of the tunnel. On the ground surface, well exposed bedrock over the upstream half of the tunnel contains only limited shearing. The junction between the two halves is a long band of colluvium which may hide a large shear or other weak zone. It is more likely that displacements on the order of fractions of inches to fractions of feet will occur during the design earthquake event on one or more distinct, but lesser, planes of weakness. It is also a possibility that small displacements, however, may occur anywhere along the outlet alignment and need not follow any distinct shear.

Reservoir Area - Landslide Potential

8-43 The literature search, aerial photo interpretation by PIC, and the geologic field mapping did not identify widespread or significant landsliding at the damsite or in the reservoir area. However, rockfalls and debris flows from the higher and steeper slopes are common. Several slip-surface slope failures were identified and mapped, especially in proximity to the North Branch fault zone and associated Tertiary sediments. As depicted on plate A-5, one such landslide was identified by Weldon (written comm.) with the scarp located at the northeast end of Government Canyon Ridge. Originally, the slide mass must have crossed and dammed Government Canyon with the stream eventually cutting through and removing the lower part of the slide mass. Minor slides within the original mass have moved into Government Canyon. The slide excavated and logged in T87-7, shown on plate A-49, is the most recent movement. The estimated total volume of remaining slide debris is only about 250,000 yd^3 and if re-activated would pose no hazard to the dam.

8-44 Another slide, an order of magnitude larger in volume, possibly exists on the north side of Santa Ana River Canyon near the Warm Springs Canyon confluence (Weldon, written comm.). Although this possible slide mass would be large, it is overlain by the oldest identified terrace surface and therefore has been stable, even under much wetter climatic conditions and higher stream levels, for on the order of 500,000 years.

8-45 It is likely that operation of the flood control dam will cause mass movements within the reservoir, most of which will probably consist of the downslope movement of saturated surficial residual soil and loose rock. No potential for large scale landsliding which would pose a hazard to the safety of the embankment was identified.

Downstream Pervious Borrow Area

8-46 The downstream pervious borrow area is located in the alluvial outwash of the Santa Ana River, from 1/2 to 3 miles downstream of the canyon mouth. It is being used by the San Bernardino Valley Water Conservation District to spread water back into the alluvial aquifer with a system of interconnected detention basins and percolation ponds. Approximately 40,000 linear feet of dikes within the borrow area make up 21 basins with a total wetted area of about 110 acres. The East Valley Water District operates a water supply well near the center of the borrow area that produces about 3,000 gal/min for domestic use. These facilities would be maintained or replaced after excavation of borrow materials. See plate A-101 for a proposed plan of relocation for the percolation basin facilities. In addition to the water agency facilities, two federally listed endangered plant species are present in the wash and impacts from borrow operations must be mitigated.

8-47 An inspection of the mass gradation curves from the five test pits excavated in the downstream pervious borrow area indicates that the materials are relatively consistent from one pit to another. A summary of the pertinent gradation values for these materials is shown in table A-51.

Table A-51. Downstream Pervious Borrow Area, Soil Investigation Summary.

TEST PIT	% -3 in.	% -12 in.	% +30 in.
TP 86-1	51	83	5
TP 86-2	41	81	0
TP 86-3	42	70	0
TP 86-4	43	78	3
TP 86-5	45	79	0
AVERAGE	44	78	2

8-48 Because of the depositional nature of alluvial outwash deposits, a gradual reduction in the quantity and size of rocks would be expected as the distance from the canyon mouth increases. This is consistent with the information gathered from investigations at the Seven Oaks damsite, which indicate rocks up to 7 feet in diameter, and at the Mentone damsite 2 miles downstream of the borrow area, which encountered only a few rocks larger than 3 feet. The maximum size was about 4 to 5 feet in the borrow area test pits, midway between the Seven Oaks and Mentone damsites. Generally, although horizontal layers indicative of variable deposition conditions were observed, no discernable difference in overall gradation with depth was obvious. Borrow excavation operations, described later, are expected to produce a relatively consistent material within the area and depth limits planned. The following paragraphs describe the various embankment fill materials that are expected to be produced from the downstream pervious borrow area.

ALLUVIAL TRANSITION MATERIAL

8-49 Relatively free-draining, high strength upstream transition materials that can be compacted to high densities can be produced from the downstream pervious borrow area by scalping out oversize rocks with a grizzley screening process. A maximum particle size of 12 inches was chosen for economy of transportation to the damsite and to reduce the tendency to segregate during placement and spreading operations. Based on gradations of the samples from the test pits, 39 to 49 percent of the minus 12-inch material would be between 3 and 12 inches in diameter. Only about 1 to 3 percent of the material would be finer than the No. 200 sieve size. The gradation curves for these materials are shown on plate A-57.

FILTER MATERIAL

8-50 Filter material to be placed adjacent to the impervious core can be produced by passing a portion of the minus 12-inch material over another grizzley screen with 3-inch openings. This would produce a

well-graded, cohesionless material with little tendency to segregate during placement and spreading operations. Gradation curves for this material are also shown on plate A-57. This material would be suitable as an upstream crackstopper zone because the cohesionless fines and subrounded sand particles could not sustain an open crack, but would instead collapse and tend to fill any crack that developed in the core due to differential settlement or earthquake motions, thus reducing seepage through the crack. This material would meet the gradation requirements for filter criteria contained in EM 1110-2-1901, "Seepage Analysis and Control for Dams." However, because the filter is well-graded and could be internally unstable, filter tests will be conducted during the FDM study phase to confirm its operating characteristics.

SHELL MATERIAL

8-51 Based on the mass gradations from the 5 test pits that were excavated in the borrow area, approximately 17 to 30 percent of the materials are larger than 12 inches in diameter. These would be produced as a byproduct of the screening operation used to obtain the upstream transition materials. Stone from 12 to 30 inches in diameter, and a relatively smaller amount of 3 to 12 inch stone from the filter material processing, would be used to construct an erosion resistant oversize shell zone on the outer slopes of the dam. The boulders larger than 30 inches, visually estimated to be less than 3 percent of the total borrow area volume, would be removed in the borrow area and wasted.

Upstream Pervious Borrow Area

8-52 The upstream pervious borrow area consists of the streambed alluvium between the upstream toe of the embankment and the confluence of Santa Ana River and Warm Springs canyons. This area is listed as an optional borrow area because of the anticipated difficulty in excavating the materials after completion of the slurry trench cutoff. Groundwater is normally within about 15 feet of the surface and is expected to quickly rise to the surface when the cutoff to bedrock is constructed. Excavation of the alluvium upstream of the cutoff will require dewatering of the streambed to allow operation by construction equipment.

8-53 The mass gradations of the two test trenches in this area that were excavated as part of the aggregate investigation indicate that the materials in the canyon are considerably coarser than in the downstream alluvial wash below the canyon mouth. Although the materials are similar, a much greater percentage of the streambed materials are larger than 30 inches in diameter, and therefore unusable in the embankment. The locations of the two trenches are shown on plate A-97 and pertinent gradations of the samples from these trenches are illustrated in table A-52.

Table A-52. Upstream Pervious Borrow Area, Soil Investigation Summary.

Trench	% -3 in.	% -12 in.	% +30 in.
TT 87-3	30	60	13
TT 87-4	27	52	19
AVERAGE	29	56	16

8-54 If utilized by the Contractor, the upstream pervious borrow area would produce free-draining, high-strength materials similar to those described from the downstream pervious borrow area. Since the volume to be excavated is considerably smaller, the construction operations would probably utilize a stationary grizzley plant with excavated materials loaded into trucks and hauled to the grizzley rather than pushed by a dozer to portable screening equipment as was estimated for the downstream borrow operation.

Impervious Borrow Area

MATERIAL DISTRIBUTION

8-55 The impervious borrow area is located in a terrace of alluvial material of granitic origin that has developed a vertical soil profile due to weathering from the surface. The weathering and resulting amount of fines is greatest at the surface and decreases rapidly with depth. The amount of weathering and soil development is roughly proportional to the amount of water that has been introduced to the soil, so that those areas with steep slopes and rapid runoff exhibit a lesser degree of soil development than those areas with gentler slopes. The investigation confirmed this pattern and identified those areas where sufficient amounts of fines are present to produce a relatively impervious core material. Generally, in the northern half of the proposed borrow area, the slopes are steep and the depth of soil with appreciable amounts of fines is relatively shallow and variable. The depth of soil with more than about 15 percent fines ranges from less than a foot to more than 20 feet. In the southern half of the borrow area, the surface is more nearly level and the soils with significant amounts of fines extend to 15 feet or more throughout the preferred area shown on plate A-60. A graphical representation of the distribution of amounts of material smaller than a No. 200 sieve is shown on plate A-61.

8-56 Generally, the impervious soils in the borrow area are silty and clayey sands with relatively few gravel and cobble size particles. Liquid limits are normally in the range between 20 and 40 percent, and plasticity indexes are mostly less than 10 percent. In the indicated

preferred area, the percent passing the No. 200 sieve ranges from 12 to 48 percent with a mean of 30 percent. Gradation curves and results of detailed laboratory tests on impervious borrow materials are shown on plate A-75.

STRESS-STRAIN CHARACTERISTICS

8-57 Because of the postulated 4-foot displacement in the foundation during a large seismic event and the anticipated differential settlement between the core and the filter zones, plastic deformation of the core materials under large strain conditions was judged to be critical to the safe performance of the embankment. Since brittleness is normally associated with low plasticity soils, a bentonite blending investigation and a detailed laboratory study of the stress-strain characteristics of the proposed core materials was undertaken.

8-58 Atterberg limits, and especially a plasticity index of 10 to 15, were used as index values of materials that were known to exhibit non-brittle deformation during large strains. The blending study showed that approximately 3 percent by dry weight of powdered bentonite was needed to significantly increase the Atterberg limits of the typical impervious borrow materials and bring the PI into the desired range. A portion of the impervious samples were blended with 3 percent bentonite and subjected to the same detailed laboratory tests that were run on non-blended samples to evaluate the effect on the engineering characteristics of the material. The results of laboratory tests on the blended materials are shown on plate A-76 and the moisture-density relationships on plate A-79.

8-59 Triaxial shear tests on impervious borrow materials compacted to 98 percent of maximum density (ASTM D 698), between minus 2 and plus 2 percent of optimum moisture content, indicate that the proposed core material does not behave in a brittle manner even when tests were continued to axial strain values of 25 to 30 percent. The stress-strain curves illustrated on plate A-77 do not show a sudden drop in strength, but rather an initial elastic type behavior followed by plastic deformation at high strains. Even a set of tests on material that was compacted to 100 percent of maximum density (ASTM D 698) exhibited plastic deformation during shear. Based on the results of these tests, and the suitability of permeability, consolidation and compactibility factors, it was concluded that blending of bentonite into the borrow materials is not necessary to obtain suitable impervious core materials.

IX. DESIGN VALUES

General

9-01 The adopted design parameters selected for the proposed embankment materials and foundation are based on conservative interpretations of the field and laboratory test results. Considerations were also given to values for similar materials on major projects constructed by the Los Angeles District and others.

9-02 The design shear strengths were selected in general accordance with the guidelines outlined in EM 1110-2-1902, "Engineering and Design, Stability of Earth and Rockfill Dams," dated 1 April 1970. The selected design values are presented in this section in table A-54.

9-03 During the FDM stage of the project, investigations for the foundation and embankment materials will include the determination of dynamic strengths and response properties. The static and dynamic performance of the materials will be analyzed using varying combinations of material densities and corresponding strengths.

Foundation

BEDROCK

9-04 The foundation bedrock at the damsite is composed of gneiss and diorite. Both rock types are generally decomposed and highly weathered adjacent to the bedrock/alluvium interface. The gneiss and diorite are highly to intensely fractured and jointed and the bedrock quality does not significantly improve with depth. Therefore, in the contact area between the core and filter zones and the bedrock, the bedrock will be excavated to a minimum depth of approximately 5 feet. Outside of this area, the bedrock will be cleaned of loose material but not excavated except where shaping is required to provide suitable surfaces for placing fill.

9-05 Unconfined compression tests of bedrock cores taken from the embankment foundation were conducted to evaluate the relative strength of the rock sample. The compressive strengths ranged from 1,790 to 13,180 lb/in^2. Triaxial compression tests evaluate the strength of the rock (intact) under conditions of lateral confinement and are useful in evaluating the foundation bearing capacity. Triaxial compression tests were not conducted for the GDM study, but will be conducted during the FDM study. For this study, conservative strength values were assumed as follows: unconfined compressive strengths of 5,000 lb/in^2 and angle of internal friction (Ø) from 35 to 40 degrees.

9-06 A lower bound bearing capacity (conservative for homogeneous, discontinuous rock masses) can be assumed as the unconfined compressive strength (5,000 lb/in^2) of the rock mass. For rock masses with a constant angle of internal friction and unconfined compressive strength, the bearing capacity can be calculated using simplified equations (Goodman, 1980). Using these equations, the ultimate bearing capacity for the dam foundation is approximately 4,000 kip/ft^2, adequate for a 650-foot high embankment (average unit weight = 145 lb/ft^3).

9-07 Shear wave velocities are used to empirically calculate elastic (E) and shear (G) moduli. Downhole and crosshole geophysical tests that measure shear wave velocities were not conducted in the bedrock during the GDM study, and will be conducted during the FDM. Based on an assumed shear velocity of 4,500 ft/s, the elastic and shear modulus were calculated as: $E = 2.0 \times 10^6$ and $G = 7.7 \times 10^5$ lb/in^2.

ALLUVIUM

9-08 The alluvial foundation materials are gravelly sands with varying amounts of subrounded cobbles and boulders. Approximately two-thirds of the alluvial foundation material consists of cobbles and boulders up to 5 feet in diameter.

9-09 Large-scale in situ density tests were performed in lenses of material with relatively few cobbles and boulders in the top 20 feet in the foundation. Based on these tests, the in situ dry unit weight of the foundation ranged from 109 to 137 lb/ft^3 with an average value of 125 lb/ft^3. This compares to the vibrated maximum density of the minus 6-inch materials at approximately 95 percent of maximum density.

9-10 The minus 2-inch materials were remolded to 95 percent of maximum density (procedure developed by the California Department of Water Resources) and tested in 12-inch diameter molds. Laboratory studies indicate that materials with a significant fraction of particles larger than 1 inch in diameter are extremely difficult to test using conventional laboratory triaxial equipment. The difficulty is that the diameter of the laboratory triaxial specimens must exceed the maximum particle size of the material by at least a 6 to 1 ratio to yield reliable results. Testing of particles larger than 2 inches exceeds SPD laboratory equipment capabilities. This problem was marginally resolved by scalping oversize particles and replacing materials with an equal

percentage by weight of material retained on the No. 4 sieve and passing the 2-inch sieve to produce samples suitable for the available equipment.

9-11 Triaxial compression (consolidated-drained) test results indicate internal friction angles ranging from 47 to 40.5 degrees for normal pressures ranging from 51 to 405 lb/in^2.

Embankment

9-12 The impervious core materials will be obtained from the Impervious Borrow Area. The rock materials for the downstream transition zone will be obtained from the Government Canyon Ridge and spillway excavation. The filter, alluvial transition, and oversize shell will be obtained from the alluvial foundation excavation and Downstream Pervious Borrow Area.

IMPERVIOUS CORE

9-13 The core materials are primarily silty and clayey sands of relatively low plasticity. The selection of design shear strengths for the core materials was determined from laboratory triaxial compression (unconsolidated-undrained, consolidated-undrained, and consolidated-drained) tests conducted on remolded impervious core materials. The samples were remolded to 98 and 100 percent of maximum density (ASTM D 698) at moisture contents ranging from 2 percent below to 1.5 percent above optimum moisture content. The samples tested were representative of the impervious core materials blended to a depth of 15 feet, based on a statistical analyses of 33 test holes in the preferred impervious borrow area. The unconsolidated-undrained, consolidated-undrained, and consolidated-drained shear strength (p-q plots) test results and K_f design envelopes are presented on plate A-75. The combined shear strength envelope used in the slope stability analysis is also presented on plate A-82. The design values are as follows:

Table A-53. Impervious Core Design Strengths.

Test Type	Angle of Friction (degrees)	Cohesion (ton/ft^2)
Unconsolidated-undrained (Q)	-	2.0
Consolidated-undrained (R)	15	0.8
Consolidated-drained (S)	36	0.0

FILTER

9-14 The filter zone would be constructed of minus 3-inch alluvial materials obtained from the downstream pervious borrow area and compacted to at least 95 percent of maximum density. The design shear strength parameters for the filter materials were determined from laboratory tests (consolidated-drained triaxial compression) conducted on remolded samples representative of minus 3-inch materials from the downstream pervious borrow area.

9-15 The minus 2-inch materials were remolded to 95 percent of maximum density (ASTM D 1557) at 1 percent above optimum moisture content and tested in 12-inch diameter specimens.

9-16 Tests indicate that internal friction angles range from 47 to 40.5 degrees for normal pressures ranging from 51 to 405 lb/in^2.

ALLUVIAL TRANSITION AND SHELL

9-17 The alluvial transition and shell materials would be obtained from the foundation excavation and the downstream pervious borrow area. As previously stated, the shell would be composed primarily of 12 to 30-inch boulders, and the alluvial transition would contain minus 12-inch materials.

9-18 Testing of these larger particles exceeds equipment capabilities. Therefore, only minus 2-inch materials (see para. 9-10, Alluvium) were remolded to 95 percent of maximum density (ASTM D 1557) at 1 percent above optimum moisture content and tested in 12-inch diameter specimens. Triaxial compression (consolidated-drained) test results indicate friction angles range from 47 to 40.5 degrees for normal pressures ranging from 51 to 405 lb/in^2.

9-19 Studies on similar alluvial materials from major projects constructed by this District and others were also considered in selecting the design strengths. Laboratory tests conducted on minus 3-inch subrounded alluvial materials compacted to medium and high densities indicated shear strengths ranging from 47 to 38 degrees for confining pressures ranging from 60 to 500 lb/in^2. In addition, shear strengths ranging from 51 to 40.5 degrees for confining pressures ranging from 10 to 500 lb/in^2 were recommended by technical expert H. Bolton Seed. A design curve was selected based on the internal friction angles and normal pressures of the laboratory tests and recommended strengths. Additional triaxial shear tests at high confining pressures will be conducted during the FDM studies to verify the design shear strengths.

DOWNSTREAM TRANSITION

9-20 Laboratory testing on the rock materials for the downstream transition zone was not performed in the GDM study since materials representative of the Government Canyon Ridge and spillway excavation

and fill could not be obtained. The test fill proposed as the initial construction start will be utilized to determine the characteristics of these materials when used as embankment fill. The test fill will subject the materials to excavation, loading, spreading and compaction procedures similar to what may be expected during construction of the dam, and thus evaluate the engineering properties of the "as placed" materials. Field tests will be used to determine gradation, particle breakdown, density and consolidation characteristics. Detailed laboratory tests on materials excavated from the test fill will be conducted in order to select final design values for shear strength and permeability. See table A-55 for a complete list of laboratory tests to be conducted on these materials during the FDM study phase. For this study, the design shear strength parameters for the downstream transition were selected from tests performed on rockfill materials by others and summarized in studies by Leps (1970). Leps' data on rockfill friction angles and their corresponding confining pressure were plotted and a design curve was conservatively selected such that two-thirds of the test values exceed the design curve.

Table A-54. Adopted Design Values.

ZONE	γ_d lb/ft^3	γ_m lb/ft^3	γ_{sat} lb/ft^3	Rphi degrees	c ton/ft^2	Sphi degrees
Impervious Core	120	136	138	20	0.8	30
Filter	125	135	140			37
Alluvial Trans.	138	145	147			41-37
Downstream Trans.	118	132	137			45-37
Shell	138	145	149			45
Alluvial Found.	133	137	146			40
Bedrock Foundation		170				40

Feature Design Testing

9-21 These adopted design values will be verified by the results of the test fill and additional laboratory testing that will be performed during the FDM study phase. Results of detailed laboratory tests such as resonant column and cyclic triaxial compression will be used to determine cyclic response chracteristics. Results of static triaxial compression tests will be used for static strengths and to determine static finite element parameters such as the deformation modulus and bulk modulus. The types of laboratory tests scheduled for the FDM study are identified in table A-55.

Table A-55. Scheduled FDM Laboratory Testing.

Laboratory Test	Zone: Impervious Core	Filter	Alluvial Transition	Downstream Transition	Shell	Alluvial Foundation
Compaction	yes	yes	yes	yes	yes	yes
Consolidation	yes	yes	-	-	-	-
Dispersion	yes	-	-	-	-	-
Permeability	yes	yes	yes	yes	yes	yes
Resonant Column	yes	yes	-	yes	-	-
Specific Gravity	yes	yes	yes	yes	yes	yes
Triaxial:						
- Static	yes	yes	yes	yes	yes	yes
- Cyclic	yes	yes	yes	yes	yes	yes
Special:						
- Filter (as per Tech Report GL-87-22)	-	yes	-	yes	-	-

X. DESIGN AND ANALYSIS

Embankment

SEISMIC CRITERIA

10-01 The seismic criteria for Seven Oaks Dam was developed by (a) defining the seismic zones, faults, and source areas, (b) developing and selecting the site or region specific attenuation characteristics, (c) calculating the site bedrock motions (acceleration, velocity, displacement, and duration), and (d) selecting analogous time-histories and response spectra, with appropriate scale factors.

MAXIMUM CREDIBLE EVENT

10-02 The maximum credible earthquake for Seven Oaks Dam is postulated as a magnitude 8+ event on the South Branch of the San Andreas fault. This fault is considered the primary seismic source capable of generating earthquakes that could cause significant ground shaking at the dam and was selected based upon the degree of activity of faulting, magnitude, peak site horizontal bedrock acceleration, duration of strong shaking, and the distance of the fault from the dam. During the FDM study, the response of the embankment and foundation to the maximum credible and maximum probable earthquake events will be determined and the stability of Seven Oaks Dam under earthquake loading will be evaluated.

SITE GROUND MOTIONS

10-03 Earthquake parameters were recommended by Bruce Bolt for the San Andreas event, see attachment A-2. The recommended seismic parameters are presented in table A-2 of section IV of this appendix. The maximum horizontal accelerations in rock based on the attenuation characteristics specific to Seven Oaks Dam, should the maximum credible and maximum probable earthquakes occur, would be 0.70g and 0.50g, respectively. These values will be applied at the base of the finite element model of the dam during the FDM dynamic analyses.

10-04 Bolt recommends a bracketed duration of 40 to 50 seconds (accelerations greater than 0.05g). However, the number of equivalent uniform cycles of motion (Seed, et. al., 1975) is most useful in soil response problems. Seed, et. al. indicates that the number of equivalent uniform cycles of motion is typically about 25 for a magnitude 8.5 event and will be used for the FDM dynamic analyses.

DESIGN EARTHQUAKE ACCELEROGRAM

10-05 The selection of the acceleration-time histories will be made after researching corrected accelerograms and selecting the one that best matches the magnitude and epicentral distance of the events to the site characteristics of Seven Oaks Dam.

10-06 No accelerograph record of ground shaking resulting from a large magnitude earthquake has ever been obtained, therefore a synthetic accelerogram must be used. Based on the seismic parameters and other seismological considerations, Bolt recommended that the maximum credible and maximum probable earthquake events be modeled by time-histories developed by ESA (in consultation with Bruce Bolt and used for seismic stability analysis of San Andreas and Pilacirtos Dams for the City of San Francisco). The FDM study will consider Bolt's accelerograph records as well as other synthetic records, such as the hypothetical accelerogram developed by Seed and Idriss (1969), known as the Seed-Idriss record. These records are shown on plate A-81.

10-07 The relative severity of the recommended design earthquake records to several other natural earthquake records will be evaluated by comparing the acceleration response spectra and the results of one-dimensional free field response analyses. Although relationships for frequency content are not as numerous as the relationships for peak accelerations, several studies are available. The relationship by Seed, Ugas, and Lysmer (1976) expressed in the form of a response spectrum has been most widely used in dam safety studies. The Seed, Ugas, and Lysmer average response spectra for different site conditions do not reflect the magnitude and distance dependencies shown in other studies, but their average spectra provide a useful reference to judge the suitability of a particular motion for use in analysis. Motions with a response spectrum similar to the Seed, Ugas, and Lysmer mean plus one standard deviation spectrum are suitable for use in conjunction with mean peak horizontal accelerations.

10-08 Acceleration response spectra for the Bolt and Seed-Idriss records representing the regional San Andreas event are presented on plate A-81. The mean plus one standard deviation record developed by Seed, Ugas, and Lysmer (1974), is presented for direct comparison. The amplification factors for the Bolt and Seed-Idriss records are approximately equal and both are higher than Seed, et. al.'s mean plus one standard deviation curve. The predominant period of the site is approximately 4 seconds. The response spectra indicates only the frequency content of a motion and the occurrence of liquefaction is very sensitive to the duration of ground shaking.

DEFENSIVE DESIGN MEASURES

10-09 Given the foregoing seismic environment, defensive measures to enable the dam to safely withstand earthquake shaking and ground movements, even under reservoir storage conditions, are included in the embankment design. These defensive design measures include:

a. Selecting an adequate freeboard to prevent overtopping due to crest settlement, uplift of reservoir bottom, or waves generated by seismic activity and landslides.

b. Using a zoned embankment section which includes a wide, deformable central core with well-compacted, free-draining upstream and downstream shells and transition and filter zones to preclude piping due to cracking.

c. Providing a well prepared foundation-embankment contact to reduce seepage.

d. Increasing the core width at the abutment contacts.

e. Using a wide and suitable armored crest to prevent erosion if overtopped.

f. Providing a conservatively designed outlet discharge capacity for lowering the reservoir quickly.

EMBANKMENT SECTION

10-10 The embankment section presented in this GDM is designed to produce a conservatively safe dam while utilizing the materials available at the site in the most cost effective manner. Internal zonation materials and slopes are based on assumed material properties and quantities in the case of materials from the spillway and drainage channel excavations, and are subject to substantial refinement after completion of the test excavation and test fill on these materials during the FDM study phase. A finite element analysis of the embankment during the FDM phase will also be used to refine the section.

Zonation

10-11 Embankment zoning for Seven Oaks Dam provides resistance to concentrated leaks by placing high strength and erosion resistant materials at the outer shells. A relatively deformable impervious core and well-graded, cohesionless filter and transition zones are designed to be effective in sealing internal cracks which may develop due to differential settlement or earthquake induced movement. The outer shells of free-draining sands, gravels and rockfill are designed with relatively flat slopes to provide stability during an earthquake.

10-12 A typical section of the proposed embankment is shown on plate A-80. The dam would be approximately 550 feet high from streambed to crest, with a crest length of 3,100 feet. The crest would be 40 feet wide and both upstream and downstream slopes would be 1V on 2H. Because the surficial streambed deposits are not as dense as the underlying materials, the top 20 feet of alluvium under the entire embankment would be removed. Within a zone defined by a 1V on 1H line from the crest, both upstream and downstream, the alluvium would be removed to bedrock. This would provide a stable central portion of the dam even under the most severe seismic shaking that can be expected. Except for the top 20 feet, the alluvium under the toes would be left in place because, below 20 feet, geophysical data indicate the streambed materials to be nearly as dense as modern construction methods could make them if they were removed and replaced. Additionally, the time and cost of excavating and backfilling the foundation would be reduced. Even though some minor sloughing of shell materials could occur during the highly unlikely event that the remaining streambed alluvium were to experience significant strength loss during a great earthquake, the dam would still be safe with this configuration.

10-13 The impervious central core would extend from bedrock to within 10 feet of the crest of the dam. The top width would be 16 feet and the base width would be approximately one-third of the maximum hydrostatic head. The core would be flanked by upstream and downstream filter zones composed of minus 3-inch alluvial materials processed from the downstream pervious borrow area or from the foundation excavation. The upstream transition would consist of free-draining minus 12-inch alluvial materials processed from the downstream pervious borrow area, with an outer layer of oversize stone. The downstream transition zone would consist of unprocessed rock excavated from the spillway and Government Canyon Ridge, with an outer layer of erosion resistant oversize stone from the pervious borrow area.

Cofferdam

10-14 A cofferdam will be constructed as a permanent part of the embankment at the upstream toe of the dam to divert surface flows into the outlet tunnel. For estimating purposes, it is assumed that a slurry trench and wells or sumps would be used to control the flow of groundwater through the site during excavation and preparation of the foundation. The cofferdam would be 50 feet high with a 20-foot wide crest, a 1V on 2H upstream slope and a 1V on 1.5H downstream slope. It would be constructed of minus 12-inch streambed alluvial materials, with a temporary impervious blanket on the upstream slope to prevent seepage of water impounded behind it and an oversize stone downstream slope to reduce erosion if overtopped. After the main embankment is raised to the elevation of the cofferdam, the temporary impervious layer will be removed and replaced with rock shell materials for erosion protection.

Slurry Trench Cutoff

10-15 The slurry trench will be excavated upstream of the toe in a location that minimizes interference with the embankment and the diversion channel. This location also reduces the required volume of

the slurry trench. It will be connected to the impervious blanket on the cofferdam with a horizontal blanket of impervious material, and will be combined with a line of wells across the upstream and downstream slopes of the foundation excavation trench, and a collector trench located along the toe of each abutment. The purpose of the slurry trench is to cut off the flow of groundwater through the foundation. The purpose of the wells on the upstream slope of the foundation excavation is to collect the groundwater that is not cut off by the slurry trench. The purpose of the wells on the downstream slope of the foundation excavation is to collect the groundwater flowing into the foundation area from downstream. The purpose of the collector trenches along the toes of the abutments is to collect groundwater seeping through the rock into the foundation area.

SLOPE STABILITY

10-16 For this GDM, the stability of Seven Oaks Dam was analyzed for static loading conditions in accordance with EM 1110-2-1902, and for limited displacement criteria. The methods of analysis and the results are described in the following paragraphs. The information from this initial analysis is preliminary in nature and subject to detailed study during the development of the Embankment FDM. Additional dynamic analyses will be conducted for the FDM in order to assess the seismic performance of the embankment.

Static

10-17 Slope stability analyses were performed for several cases of static loading conditions in general accordance with EM 1110-2-1902, "Engineering and Design, Stability of Earth and Rockfill Dams," dated April 1, 1970. The UTEXAS2 computer program was used in conjunction with graphical studies to locate potentially critical surfaces of sliding in the embankment, and to confirm the results of manual calculations. The manual solutions utilized general force equilibrium equations for non-circular sliding surfaces as outlined by Duncan and Buchignani in "An Engineering Manual For Slope Stability Studies." The UTEXAS2 computer program utilized Spencer's procedure (1967) in a force and moment equilibrium solution for non-circular surfaces of sliding. The computer calculated results of the cases that were analyzed are shown below in table A-56 and presented graphically on plates A-82 through A-89.

Table A-56. Computed Safety Factors for Static Loading.

Loading Condition	Computed FS	Minimum Allowable FS
End of Construction (upstream)	2.4	1.3
End of Construction (downstream)	2.1	1.3
Sudden Drawdown (from max. pool)	2.3	1.0
Partial Pool	2.1	1.5
Steady Seepage (with max. storage pool)	2.1	1.5

10-18 The design assumptions and methods of analysis for each of the loading conditions are discussed in more detail in the following paragraphs.

End of Construction

10-19 This case represents a short-term loading condition which evaluates the effects of pore pressure on the undrained shear strength of soil. The development of pore pressures in a compacted cohesive soil depends primarily on the placement water content, density, and applied total stresses. Instrumentation will be installed to monitor pore pressures and the rate of placement of embankment materials will be regulated, especially within the core zone, to maintain embankment stability and to limit development of excess pore pressures. Given the types of materials to be placed, the range of moisture contents to be specified for placement, the construction period, and the climatic conditions at the project site, it is highly improbable that excessive pore water pressures would develop in any of the embankment zones. Total stress strength parameters were used in this analysis, the results of which indicate safety factors of 2.4 for the upstream slope and 2.1 for the downstream slope.

Sudden Drawdown

10-20 During this loading condition, it is assumed that the upstream embankment is completely saturated at a maximum pool elevation and then subjected to a rapid draw down in pool elevation. The upstream slope is then analyzed with an elevated phreatic surface. The embankment materials are generally free draining thereby avoiding a build up of excess pore pressure and unbalanced seepage forces. Drained shear strengths were used for all of the embankment zone materials except for the core zone which was defined by a combined "R" and "S" shear strength envelope (for conditions of high normal stresses, the "R" shear strength envelope was used). The water surface profile shown on plate A-87 has very little effect on the critical sliding surface. These parameters produced a safety factor of 2.3 for the sudden drawdown condition.

Partial Pool

10-21 The partial pool analysis of the upstream slope consisted of a search for the lowest factor of safety at different pool elevations. These pool elevations were considered tc be at equilibrium with a steady seepage phreatic surface. The embankment materials strengths were defined by "S" shear strength envelopes with the exception of the finer grained core material which was defined by a "(R + S)/2" shear strength envelope. A pool elevation of 2,355 ft. was determined to be most critical with a safety factor of 2.1 for the upstream slope. The critical sliding surface is shown on plate A-88.

Steady Seepage

10-22 Analysis of the downstream slope for the steady seepage condition with a maximum storage pool elevation was determined to be the same as the end of construction case since seepage forces were not present along

the critical sliding surface. The free draining materials used in the embankment and the low phreatic surface indicate that seepage forces do not effect the stability of the slope and the computed factor of safety of 2.1 is considered applicable for this case.

Earthquake

10-23 The project is located within Zone 4 of the Seismic Risk Zone Map, having a recommended seismic coefficient of 0.15. The seismic force, developed from the weight of the moving body, is a lateral load acting in the direction of the potential sliding. Although the seismic coefficient method prescribed in EM 1110-2-1902 is not intended to be a definitive evaluation of a high embankment located in a highly seismic area, such as Seven Oaks Dam, limited time and funds for this GDM dictated that the analyses be performed as a preliminary check on seismic stability. The results are presented here for comparison with similar analyses on other projects only. Detailed dynamic response and permanent deformation analyses, as described later, will be conducted in the FDM study phase when funds and dynamic test results are available. Pseudo-static slope stability analyses were performed for the following cases; End of Construction (upstream and downstream), Partial Pool (upstream), and Steady Seepage (downstream) with phreatic surface based on a maximum pool elevation. Strength parameters and soil properties for seismic analyses remained the same as the non-seismic evaluations. Factors of safety for seismic analyses are listed below, and are also shown on plates A-84, A-86 and A-89 with the subscript "Q" to designate the earthquake loading.

Table A-57. Computed Safety Factors for Seismic Loading.

Loading Condition	Computed FS	Minimum Allowable FS
End of Construction (Upstream)	1.8	1.0
End of Construction (Downstream)	1.5	1.0
Partial Pool	1.4	1.0
Steady Seepage (with max. storage pool)	1.5	1.0

Finite Element Method Analysis

10-24 The behavior of an earth/rockfill dam during an earthquake is significantly influenced by the static stress conditions existing in the embankment and foundation prior to the earthquake event. During the FDM study phase, the proposed embankment will be analyzed using a finite element method computer program to determine the pre-earthquake stress distribution (vertical, horizontal, and shear) within the embankment and foundation. Stresses and displacements in the embankment cross section due to gravity loads and seepage forces will be determined through a

sequence of steps that simulate construction and reservoir filling. Reservoir effects will be simulated for end of construction, intermediate storage, and spillway crest conditions. The results of the analysis will be used in the dynamic seismic stability analyses and as a basis for the development of an instrumentation plan for the dam. Instruments that measure earth movements will be located in areas within the embankment where the analyses indicates that differential movements between zones of different materials may occur.

Dynamic

10-25 The principal purpose for performing dynamic response analyses is to calculate the dynamic shear stresses in the embankment that would be induced by a large earthquake. The analysis of Seven Oaks Dam under dynamic loading conditions will consist of evaluating the embankment response to a maximum credible earthquake event under dry and intermediate storage reservoir conditions and to a maximum probable earthquake event under spillway crest condition. The second purpose is to perform permanent deformation analyses. During the FDM phase, these analyses will be performed as described in the following paragraphs.

Finite Element Method Analysis

10-26 The stability of Seven Oaks Dam under earthquake loading will be evaluated during the FDM study phase using a dynamic analysis procedure developed by Seed, Lee, and Idriss (1969). This procedure (Seed, 1973) involves the following steps:

1. Use static finite element computer program to calculate the intial stress conditions in the embankment and foundation prior to the earthquake event.

2. Determine the characteristics of the motion developed in the rock underlying the embankment and its foundation during the earthquake.

3. Use dynamic finite element computer program to compute the response of the embankment and foundation to the base motion, to determine the earthquake-induced stresses throughout.

4. Determine the effects of the earthquake-induced stresses on soil elements in the embankment by subjecting representative soils samples to cyclic loading.

5. Evaluate the overall deformation and stability of the embankment and foundation from the knowledge of the potential deformation of the individual elements determined in step (4).

The embankment and foundation conditions will be modeled using the information obtained from design and test fill investigations. Dynamic properties will be based on laboratory cyclic shear tests and geophysical field testing. An idealized section using conservative assumptions will be used to model the dam.

Permanent Deformation Analyses

10-27 Due to the limitations inherent in a pseudo-static slope stability analysis to quantitatively predict the performance of an embankment during an earthquake, a simplified analysis was conducted to estimate the permanent earthquake-induced deformation. This simplified method is based on the dynamic displacement analysis procedure originally proposed by Newmark (1963) and subsequently modified by other investigators such as Makdisi and Seed (1977) to incorporate knowledge of the dynamic response of embankments and the strain dependent, non-linear behavior of soils during cyclic loading. The procedure assumes that sliding occurs along a well defined plane and that the soil behaves elastically below a yield stress level and plastically above. The procedure has been recommended by the Committee on Earthquakes of the International Commission on Large Dams (ICOLD, 1975) and has been used in the design of many large dams around the world. The ability of this method to predict the permanent displacements due to earthquake shaking for a very large dam in a highly seismic environment is not known with a high level of certainty due to the relative lack of information concerning such a combination of factors. However, for the limited scope of this GDM, the method may be considered to reasonably approximate the potential deformations and, thus, form a preliminary basis to evaluate the relative significance of such an event on the performance of the dam.

10-28 The permanent displacement of an embankment is primarily influenced by its first natural period and the maximum horizontal acceleration applied at the embankment base. The first natural period of the embankment was estimated using the procedure proposed by Makdisi and Seed (1977).

10-29 Maximum ground surface accelerations were determined using the computer program SHAKE, developed in 1972 by Schnabel, Lysmer, and Seed, University of California, Berkeley. The SHAKE computer program computes the response of one-dimensional horizontal soil deposits to base rock motion. The program uses wave propagation theory and incorporates nonlinear soil behavior, the effect of the elasticity of the base rock and systems with variable damping. Using the SHAKE model, maximum ground surface (i.e., existing streambed elevation) accelerations of 1.25g were calculated for the maximum credible earthquake event given base rock motions with maximum horizontal acceleration of 0.7g.

10-30 From the conservative maximum ground surface acceleration of 1.25g at the base of the dam, an average acceleration value of 0.83g for the critical sliding mass in the embankment was determined using a procedure proposed by Makdisi and Seed (1977). A yield acceleration value of 0.5g for the critical sliding mass was determined using a static, limit equilibrium solution defined by Spencer's procedure as employed in the UTEXAS2 computer program. Using these parameters the estimated permanent displacements induced by the design earthquake ranged from 1.6 to 2.7 feet. Deformations of this magnitude would not significantly effect the performance of Seven Oaks Dam which is a flood control dam and has an inordinate amount of freeboard, even under SPF or spillway flow conditions.

10-31 A recent Waterways Experiment Station report entitled "Rationalizing the Seismic Coefficient Method", (WES Miscellaneous Paper GL-84-13) established a procedure based upon a review of the pseudo-static and dynamic displacement methods which proposes the use of the pseudo-static analysis to determine the necessity for a detailed dynamic displacement analysis. This procedure recommends the use of a seismic coefficient equal to one-half the predicted peak bedrock acceleration and a 20 percent reduction in the soil shear strengths. If the required minimum allowable factor of safety of 1.0 is satisfied, the procedure suggests that permanent deformations will be minor and that a detailed dynamic displacement analysis need not be performed. Following the WES procedure, the calculated factors of safety for the end-of-construction condition with earthquake loading were determined and are presented below in table A-58. As previously mentioned for the dynamic displacement analysis, the applicability of this procedure to a highly seismic area is not known with great certainty and is included here only for informational purposes. Detailed dynamic response and permanent deformation analyses, as previously discussed, will be conducted during the FDM study phase to satisfy the requirements of ER 1110-2-1806.

Table A-58. Computed Safety Factors for Seismic Loading Based on WES Procedure (WES Misc. Paper GL-84-13).

Loading Condition	Computed FS (1V on 2H)	Minimum Allowable FS
End of Construction (upstream)	1.2	1.0
End of Construction (downstream)	1.1	1.0

SETTLEMENT

10-32 Consolidation test results, performed in accordance with EM 1110-2-1906 on impervious borrow materials compacted to 98 percent of maximum density (ASTM D 698), were chosen to represent the settlement characteristics of the impervious core when compacted using the proposed construction procedures. The amount of settlement was calculated over the range of water contents to be specified to give a range of probable values.

Total Settlement

10-33 Total settlement of the embankment was calculated for the impervious core at the maximum section in order to determine the largest expected settlement. Due to the extreme height of the dam, it was recognized that a straight line triangular distribution of vertical pressure would not accurately represent the effective pressures used to calculate the settlement within the core of the embankment. Instead, charts developed by Perloff (see pl. A-91), giving influence diagrams

for vertical normal stress along selected sections ($u = 0.3, \alpha = 30$, $L/H = \infty$) were used in the settlement calculations. The total settlement of the maximum embankment core zone at optimum water content and at optimum plus 3 percent water content was calculated to be approximately 20 feet, of which 18 feet would occur during construction. Based on relationships between predicted and measured settlements at other large dams, however, the actual settlement is expected to be much less than 20 feet.

10-34 Post-construction settlement of the embankment core zone was estimated to be about 2 feet. Due to the length of the construction period, most of the settlement in the core zone (approximately 95 percent) occurs during construction. The calculated settlement of the impervious central core is shown graphically on plate A-91.

10-35 The filter, transition and shell zones would consist of granular materials. Settlement in granular materials would be less than for the impervious core materials and is also assumed to occur during construction.

Differential Settlement

10-36 Differential settlement between the impervious core and filter zones will occur because of the different consolidation characteristics of the two materials and the height of the dam. Horizontal cracks in the core zone are thus a possibility. The extent of cracking and actual differential settlement is expected to be relatively small, however, because (a) the core will be well compacted, (b) high overburden stresses will tend to close cracks in all but the upper portions of the dam, (c) the 6V on 1H outer slopes of the core zone combined with the cohesionless filter materials will tend to move together instead of separating, and (d) the relatively wide core will preclude continuous horizontal cracks. The potential for differential settlements between the impervious central core materials and the materials of the adjacent filter zones will be evaluated using static finite element analysis during the FDM study phase. In addition, the variability of the abutment slopes and the effects of abutment shaping within the core contact area will be evaluated.

Foundation

10-37 To assure a dense embankment foundation, the top 20 feet of alluvium would be removed prior to construction of the embankment and then replaced and compacted as dense as practicable. The materials below about 20 feet are very dense. Settlement of these materials would occur during construction with minimal post-construction or differential settlement occurring thereafter.

SEEPAGE

10-38 Seepage through Seven Oaks Dam was estimated according to the principles in EM 1110-2-1901, "Seepage Analysis and Control for Dams". The results of the seepage analysis are presented on plates A-92 through A-95, and discussed in the following paragraphs.

Embankment

10-39 Two reservoir levels and two embankment conditions were analyzed to evaluate three possible cases of seepage in the embankment. The three cases analyzed are as follows:

Case I. Steady-State Seepage with the Reservoir Pool at Spillway Crest Elevation (Intact Core)

Case II. Steady-State Seepage with the Reservoir Pool at Spillway Crest Elevation (Cracked Core)

Case III. Steady-State Seepage with the Reservoir at Debris Pool Elevation

10-40 These analyses are conducted for academic considerations of possible emergency situations or future water storage conditions. Cases I and II represent emergency situations which are assumed to develop under the extremely improbable occurrence of a maximum probable flood during which the outlet works become inoperable. Case III models a possible future storage condition designed to prevent the deposition of sediments and debris near the upstream toe of the dam and the outlet works.

10-41 The hydraulic conductivity (permeability) values used in the seepage analyses were conservatively selected based on laboratory test results and experience with similar materials in the project vicinity. Permeability tests performed in the laboratory measured fluid flow through soil samples in the vertical direction. Effective hydraulic conductivites were determined using assumed relationships between the horizontal and vertical flow directions appropriate for each material type. The calculated effective hydraulic conductivity values are presented in table A-59. Given the constraints of accurately measuring the hydraulic conductivity of a soil, for the purpose of the analysis, the filter and both transition zone materials were conservatively considered as a single, homogeneous section possessing an effective hydraulic conductivity of 70 ft/day. The hydraulic characteristics of the downstream transition materials will be further investigated during the test fill contract and seepage reevaluated if necessary during the FDM study phase. The zone V shell materials are the most pervious materials in the embankment and were conservatively assumed not to effect seepage through the embankment.

Table A-59. Effective Hydraulic Conductivity Values.

Material Type	Effective Hydraulic Conductivity, K'(ft/day)
Zone I - Core	0.1
Zone II - Filter	70.0
Zone III - Alluvial Transition	180.0
Zone IV - Downstream Transition	180.0
Zone V - Shell	3,000.0

Case I: Steady-State Seepage with the Reservoir Pool at Spillway Crest Elevation (Intact Core)

10-42 Seepage through the embankment is designed to be controlled by the zone I materials and is not expected to occur under the proposed flood control operation of Seven Oaks Dam due to the relatively short (less than 13 days) reservoir pool detention time. However, to obtain seepage quantities under steady-state conditions, the reservoir pool was assumed to be maintained at the spillway crest elevation for an indefinite period of time. The estimated seepage quantity for the conditions stated above was determined from the flow net shown on plate A-92. It is estimated that for a maximum section, an approximate seepage quantity of 290 ft^3/day/foot of embankment would result under the assumed conditions.

Case II: Steady-State Seepage with the Reservoir Pool at Spillway Crest Elevation (Cracked Core)

10-43 Given the improbable conditions that the reservoir is maintained at spillway crest elevation and the entire core zone is cracked, steady-state seepage would be established in the embankment. Under these conditions, the cracks in the core zone were assumed to be filled by adjacent materials and the embankment, with the exception of the outer shell materials, was thus analyzed as a homogeneous section possessing an effective hydraulic conductivity of 70 ft/day. A flow net was constructed based on these assumptions in order to estimate seepage quantities, seepage forces, and the location of the discharge water surface. The results of this analysis are presented on plate A-93.

Case III: Seepage with the Reservoir at Debris Pool Elevation

10-44 As previously stated, seepage through the embankment would be controlled by the zone I core materials. Should the reservoir pool level be maintained at the debris pool elevation, more than 13 days would be required for the phreatic surface to penetrate the core zone and develop a steady-state seepage condition. Under standard project flood operation, impoundment above this pool elevation would last for a

period of about 8 days and thus, steady-state seepage conditions would not be established at any higher elevation. However, the steady-state seepage condition was assumed to exist in order to evaluate seepage quantites, seepage forces, and the location of the discharge water surface. The estimated seepage quantity for this case was determined from the flow net shown on plate A-94 to be about 35 ft^3/day/foot of embankment.

Foundation

10-45 Underseepage at the proposed embankment would be relatively small due to the low hydraulic conductivity characteristics of the diorite and gneiss foundation bedrock. In situ hydraulic conductivity values of the foundation rock, as measured in the horizontal direction by packer tests, ranged from 0.01 to 0.32 ft/day. These values are on the same order of magnitude as the value assumed for the zone I core materials. The underseepage would likely be further reduced by pressure grouting the bedrock under the core zone. Zones I and II materials would be placed directly upon the cleaned and surface treated bedrock.

10-46 Based on the operation plan, which would maintain a debris pool at about elevation 2,200, the quantity of underseepage was calculated to be on the order of 25 ft^3/day/foot width of the foundation at the maximum section. Since standard project flood operation indicates that water would be impounded above the debris pool elevation for only about 8 days, a significant increase in the steady underseepage is not expected to occur.

SLOPE PROTECTION

10-47 Slope protection for erosion and/or wave runup on both the upstream and downstream slopes will be provided by the outer zone of oversize stone that is processed from foundation excavation and alluvial borrow sources. The stone in this layer will vary from 12 to 30 inches in diameter, and, to utilize the amount of oversize stone that is produced, the layer thickness will vary from 12 feet at the top of the embankment to approximately 100 feet at the downstream toe and 300 feet at the upstream toe.

10-48 Hydraulic requirements for minimum revetment design are based generally on the criteria in EM 1110-2-2300, "Earth and Rock-Fill Dams, General Design and Construction Considerations." The analysis according to Appendix C of the EM indicates that two zones have hydraulic requirements for riprap based on estimates of wave height and runup. These are summarized in table A-60.

Table A-60. Minimum Revetment Design.

Class	Top Elev. (feet)	Layer Thickness (inches)	Maximum Stone Diameter (inches)	Median Stone Diameter (inches)	Minimum Stone Diameter (inches)
I	2,372	36	36	23	12
II	2,584	27	27	17	9

10-49 Except for the maximum stone diameter for class I riprap, the criteria shown in table A-60 is more than adequately met by the outer zone of oversize shell material. Additional stone to 36 inch diameter is available and can be placed in the outer few feet of the shell to meet that requirement.

Spillway

CUT-SLOPE CONFIGURATION

10-50 The spillway cut-slopes were established on the basis of conservative geotechnical and engineering judgement. Unoriented discontinuity frequency data from core holes drilled for the Phase II GDM indicates a preference for 60 degrees dip followed by 40 and 45 degrees. Since many of the discontinuities showed indications of shearing, it is likely that if the orientation of the fractures are adverse to the spillway alignment, the cut slopes as shown could be unstable. It is felt, however, that such occurrences will be localized. The cut-slope configuration selected, with 25-foot wide benches, 50 foot bench height, and 1V:0.5H maximum slopes, will allow for flattening of the slopes if necessary during FDM design or construction without significantly affecting the volume or aerial extent of the excavation. Additional work during the Embankment FDM study phase, including video borehole surveys and detailed surface discontinuity mapping will allow further refinement of the cut-slope design prior to plans and specifications.

INVERT SILL

10-51 Even though there is only a remote possibility of a spillway flow during the project life, the spillway invert sill was moved approximately 300 feet upstream from its location in the Phase I GDM Supplement to give added protection against headcutting erosion beginning at the downstream end of the spillway channel. In addition, the sill was increased in size. The 10-foot deep sill will likely require controlled blasting techniques for excavation. Cut slopes in the trapezoidal shaped excavation will be 1V on 0.25H.

Drainage Channel

10-52 Because of the relatively narrow bottom width (30 feet) and the somewhat adverse orientation of the foliation in the gneissic rockmass (see para. VIII-30), the cut slopes in the drainage channel will be flatter than in the spillway. Slopes will be 1V on 1H with 15-foot wide benches. Slope height between benches will be 75 feet. The slopes as well as the floor of the excavation will be unlined. Slope reinforcement or stabilization measures will be utilized on an as-needed basis during construction.

Outlet Works

PORTALS

10-53 The outlet works will include large control structures to regulate the flows through it. Depending on the final location of the control (inlet, mid-tunnel or at the outlet portal), large excavations into the bedrock would be required. All concepts include a 200-foot-plus high intake tower with a high cut at the inlet portal. The outlet portal will also require a cut in rock at least two tunnel diameters over the tunnel. See Volume 1, Appendix B for details of the three control alternatives.

Cut-Slope Configuration

10-54 At the inlet, the intake tower is located so that the excavation subgrade is just inside the alluvial/bedrock contact at elevation 2,080. This location will minimize rock excavation both in the floor and the portal face. Some adjustment may be necessary in the Outlet Works FDM when the subsurface bedrock configuration is refined and also to allow for a gate structure if upstream control is selected. Since the bedrock is tight, little dewatering is anticipated for the intake foundation. The portal face will be sloped 1V on 0.25H to the tower subgrade. The side slopes will also be 1V on 0.25H except those in the tower foundation which will be near vertical. Determination of the portal contours depends on better resolution of the ground contours and the final intake design. However, should the cut daylight into the natural slope above more than three tunnel diameters over the tunnel entrance, a 20-foot wide bench will be included. At the outlet, the same general design features as at the inlet will apply. The colluvial overburden would be completely removed, probably down to the top of the exit channel. A 20-foot wide bench may be constructed over the portal to capture stray rock falls from above.

Slope Reinforcement

10-55 The bedrock at both portal cuts will be stabilized where necessary with rock bolts and wire mesh and shotcrete. Depending on the final selected design, the wire mesh and shotcrete may be substituted with thicker reinforced concrete at either portal. A tentative rock

bolt design has been proposed by the Portland District in Volume 1, Appendix B. Adjustments to the design may be made after additional investigations during the Outlet Works FDM study phase.

Drains

10-56 Because the bedrock is apparently tight, few drains will be provided into the rock at either portal face. Inclusion of this feature, however, will be contingent on future investigations.

TUNNEL

10-57 The tunnel will be approximately 1,700 feet long excavated totally in diorite except for possibly very occasionally gneissic rock. The gradient will be approximately 3 percent, dropping from an elevation of 2,100 at the inlet to 2,065 at the outlet. Further details of the tunnel design are presented in Volume 1, Appendix B.

Configuration

10-58 The general tunnel configuration will be horseshoe shaped regardless if an isolated metal penstock is installed for downstream control. In this case, the tunnel would be smaller and used solely for diversion during construction of the dam. For mid-tunnel control, a vertical access shaft with a gate chamber would be constructed in an area where it is judged that larger displacements would least likely occur. For control at the tunnel inlet, the horseshoe shaped tunnel would be heightened to provide space for an overhead access gallery from the downstream end.

Support

10-59 Regardless of the tunnel use, it will be fully supported throughout to contain the blocky and seamy rock. Based on very limited data, it is anticipated that metal supports and rock bolts will be used and supplemented with a concrete lining. For mid-tunnel control, exotic support may be required in the gate chamber excavation.

Temporary

10-60 Because of the very blocky and seamy rock conditions, the excavation "stand-up time" will be relatively short and support, consisting of H-shaped metal ribs, will be installed contemporaneous with the excavation. Based on the experience with the tunnel at Mojave Dam on the opposite side of the San Bernardino Mountains and in very similar rock, rib spacing will likely average 4 feet. The ribs will be tied together with spacer rods and blocked. Rock bolts and shotcrete will be added where necessary for additional support.

Permanent

10-61 To allow for smooth flow during diversion and for additional support, the tunnel will be fully lined with concrete and critical areas in the upstream portion will be reinforced. Following placement of the lining, voids remaining between the lining and the rock in the crown will be backfilled by contact grouting.

Rupture

10-62 Given the hypothesis that a maximum 4 feet of displacement has to be considered on one shear crossing the outlet, the tunnel needs to be designed to accommodate this 4 feet without affecting the stability or safety of the dam and still be able to drain the pool for repairs. If that displacement, or near to it, should occur on a single plane, that plane would be a significant shear already in existence. Although a number of these planes could exist, they should be identifiable. In the tunnel support, the design could be adjusted so that the tunnel could rupture without materially damaging the support or lining. This could be accomplished by methods such as slip joints or un-keyed construction joints in the lining. However, any displacement would likely be distributed unequally among shears of all sizes including those only a hair wide and several inches apart. Therefore, the displacement literally could occur along any number of planes of weakness and in any amount up to a cumulative total not to exceed 4 feet through the site.

Consolidation Grouting

10-63 It is possible that, at time of high pools, water may enter the ridge above the tunnel and impose a hydrostatic loading on the tunnel even though the bedrock appears to be tight. The loading would be most critical upstream of the dam centerline. In order to reduce the loading potential, grout rings and drains will be installed. Grout rings will consist of holes drilled 20 feet into rock, or to the dam grout curtain, where applicable. The holes will be located fully around the tunnel in a "fan" pattern 30 degrees apart (12 holes). The number and locations of the rings will be determined during the FDM design. Drains will be 3-inch diameter holes drilled 20 feet into rock and spaced 10 feet apart along the upstream side of the dam axis.

CHANNELS AND ENERGY DISSIPATOR

10-64 The approach channel will be relatively long compared with the exit channel and energy dissipator (plunge pool). The approach is designed to approximate the natural local gradient of the Santa Ana River.

Configuration

10-65 Most of the approach channel will be excavated through alluvium where the cut slopes will be 1V on 2H. It is anticipated that more natural slopes will develop during initial floodflows. For the exit

channel, however, even though it will also be excavated through alluvium, the slopes can be steeper since they will be backfilled once the channel walls are constructed. Part of the outlet channel adjacent to the tunnel will be excavated through bedrock, in which case the cut slopes will be near vertical. The side slopes of the energy dissipator, also in alluvium, will be excavated 1V on 2H.

Slope Protection

10-66 At this time, it is anticipated that the inlet channel will be unprotected and that the outlet channel will be concrete lined. The energy dissipator, however, will be armored with oversize boulders from the streambed alluvium. Based on the experience with the plunge pool at Mojave River Dam, the excavated slopes will be covered with stones of adequate size to withstand the design maximum and lesser flows.

Summary

10-67 Based on the limited investigations, testing and analysis to date, the foundation conditions, construction materials and embankment design of Seven Oaks Dam appear suitable for the intended purpose of flood control on the Santa Ana River. Additional subsurface investigations of the foundation, abutments and spillway will be performed as described in paragraphs 5-62 through 5-71 to verify conditions presented in this GDM. Video borehole cameras will be used in test holes drilled in the spillway to attempt to determine fracture and joint patterns that could affect the slope design of the excavated spillway. The engineering properties of construction materials for the dynamic analysis of the embankment will be determined with additional static and dynamic laboratory tests as shown in table A-55, page A-IX-6. Preliminary static stability analyses, though not conclusive, do not indicate potential static instability. A state-of-the-art dynamic response and permanent deformation analysis will be conducted during the FDM study phase, as described in paragraphs 10-24 through 10-26, in order to predict the performance of the dam during expected seismic activity.

XI. CONSTRUCTION MATERIALS

Embankment Materials

11-01 Materials required for construction of Seven Oaks Dam are available from required excavation and borrow areas in the vicinity of the dam as described below. The estimated quantities and distribution of materials from each source, including waste, are shown on plate A-99.

CORE

11-02 An estimated 4,633,000 yd^3 of impervious material will be required for construction of the core and slurry trench cutoff for Seven Oaks Dam. Suitable materials are available from the impervious borrow site next to Mill Creek. Approximately 9,000,000 yd^3 could be obtained from the preferred area within the borrow area shown on plate A-60 if this area were excavated to an average depth of 15 feet. Based on comparison of in situ and maximum density tests, a balance factor of 0.90 should be applied to materials from this area to calculate embankment quantities.

FILTER

11-03 Approximately 2,656,000 yd^3 of filter material will be required as upstream crackstopper and downstream filter zones adjacent to the core in the embankment. These materials can be produced by processing alluvial materials from the downstream pervious borrow area, the upstream streambed borrow area, or the foundation excavation to produce minus 3-inch sands and gravels.

ALLUVIAL TRANSITION

11-04 Approximately 10,601,000 yd^3 of alluvial transition materials are required to construct the majority of the upstream pervious shell of the embankment. These materials would be obtained from the required foundation excavation, the upstream streambed borrow area, and, mainly, from the downstream pervious borrow area. Assuming an average depth of

excavation of 40 feet, approximately 350 acres would be required in the downstream pervious borrow area. The preferred location of this borrow area is shown on plate A-55. The materials from these borrow areas would be processed to produce minus 12-inch materials for the rockfill transition zone.

DOWNSTREAM TRANSITION

11-05 An estimated 9,000,000 yd^3 of downstream transition materials will be produced from required excavation of the spillway and the drainage channel through Government Canyon Ridge. Pending completion of the test excavation and test fill contract, it is assumed that the excavated materials will not need to be processed to produce suitable fill materials. A bulking factor of 1.15 has been assumed for estimating purposes. Due to anticipated cost savings compared to alluvial materials, overexcavation of Government Canyon Ridge is currently planned in order to produce enough additional materials to construct the entire downstream transition zone. The total in-place volume of the zone would be 13,155,000 yd^3.

SHELL

11-06 Excavation of the foundation and alluvial borrow areas to produce the required quantities of filter and upstream transition materials would leave, as a byproduct of the screening process, approximately 7,454,000 yd^3 of oversize rock from 3 to 30 inches in diameter which would be used in the outermost zones of the embankment. Stones larger than 30 inches in diameter would be placed in the outlet works energy dissipator or wasted in the borrow areas. A bulking factor of 1.20 has been assumed to estimate the volume of rock in place in the embankment. To improve trafficability on the oversize shell materials, it is estimated that about 745,000 yd^3, or 10 percent of the shell volume, of smaller material would be required to fill the voids on the surface of the rock shell lifts so that trucks would be able to drive on them. The use of alluvial, spillway and abutment excavation materials for this purpose will be evaluated in the test excavation and test fill contract during the FDM study phase.

Concrete Materials

GENERAL

11-07 The regulating outlet (RO) features for the Seven Oaks Dam of the Santa Ana Project encompass the large majority of concrete to be used in the project. The outlet works for this dam are located on and through the left abutment. The system is comprised of a high level intake tower, control gates, a regulating outlet/diversion access tunnel, a downstream access and equipment structure, and an outlet channel connecting the tunnel portal and an energy dissipating plunge pool. Based on the downstream control alternative, the following table summarizes approximate quantities of concrete and grout to be used in construction of the outlet works.

Table A-61. Estimated Concrete Quantities.

Feature	Shotcrete (cy)	Concrete (cy)
Tunnel Excavation	1,800	
Intake Structure		
(1) Below Elev. 2156		8,700
(2) Above Elev. 2156		4,600
Access Bridge		L.S.
Downstream Control Structure		2,500
Channel and Flip Bucket		2,000
Tunnel Floor and Walls		6,500
Conduit Support		6,500
Cutoff Wall		500

CLIMATIC CONDITIONS

11-08 The climate of the Seven Oaks Dam drainage area is subtropical semi-arid with warm summers and relatively mild winters. From late spring through mid-fall, with the greatest intensities during the summer, the channel area is subject to air pollution accumulation between the late morning and early evening. Variations in the climate are almost entirely due to seasonal changes.

11-09 The area is generally mild, free from extremely low winter temperatures and relatively immune to extremely high summer temperatures. Summers are pleasantly warm, with daily maximum temperatures averaging around 95°F (extreme highs around 115°F) and a nocturnal minimum ranging from 36°F to near 58°F. Winters are cool, with mild days. Normal daily temperatures range from highs of 68°F to lows of 37°F (with all-time extremes from 17° to 93°F).

11-10 The relative humidity in the project area can vary from zero to 100 percent. Typical ranges during most of the year are from 80 to 90 percent during the early morning hours to 30 to 50 percent during the early afternoon.

11-11 Mean annual precipitation in the region is approximately 18 inches. The primary rainy season is winter (November-April), with the heaviest precipitation occurring normally from December through mid-March. Summer (June-September) is the driest time of the year. More detailed information will be available in the Feature Design Memorandum for the Outlet Works.

SOURCES OF CEMENT

11-12 The following sources are producing cements which are being tested and reported in accordance with the requirements of ER 1110-1-2002, Cement, Pozzolan, and Slag Acceptance Testing. Cement would be available from the California Materials Company (CALMAT) plants at either Colton or Mojave, California; or from the Riverside Portland Cement Company at Oro Grande, California.

SOURCES OF POZZOLANS

11-13 A variety of pozzolans are available in the region surrounding the Seven Oaks Dam construction. In conformance with Environmental Protection Agency (EPA) regulations all Federal agencies utilizing Federal funds for construction are required to allow the use of the flyash in concrete unless the use can be shown to be technically improper. Flyashes are specified in conformance to ASTM Standard C 618. Class 'F' ashes are available and have been extensively used in construction throughout the region. The Class 'C' ashes have recently been introduced to the region and are developing some use. Although Class 'N' pozzolans, volcanic ashes and pumice, have been imported into the region in the past; their use has been minimal. Recently, interest has been shown in the use of Class 'C' flyashes in southern California and Arizona. The reason for this interest is the influx of Class 'C' ashes primarily from Texas with additional sources in Wyoming.

11-14 In evaluating the suitability of use of Class 'C' flyashes EM 1110-2-2000 raises some questions. Investigations to determine the suitability of Class 'C' ashes must resolve the following points.

1. **Variability.** The fly ash produced from any industrial burning of coal will vary depending on load factors, emission control techniques and boiler efficiency. These variations may occur on an hour by hour basis and variability is exacerbated by the high degree of variability of the lignite and subbituminous coals.

2. **Alkali Content.** The alkali content of available class C pozzolans should be always determined in accordance with CRD-C 255 (ASTM C 618), if class C is specified. There are pozzolans of this class which contain enough readily available alkali to combine with low alkali cement to produce an aggregate reaction when one would not have occurred otherwise.

3. **Heat of Hydration.** Materials meeting the requirements of class C pozzolan have been tested which exhibit the tendencies of a very fast setting hydraulic cement. When water is added to this material it sets very rapidly and produces substantial heat of hydration.

4. **Resistance to Sulfate Attack.** Some Class C pozzolan has demonstrated a low resistance to sulfate attack and should not be used in areas of sulfate exposures.

AGGREGATES

11-15 Aggregates for concrete construction of the dam may come from a variety of sources. These sources should include developed existing commercial sources within proximity of the dam and excavations on site.

Off-Site Sources

11-16 The Waterways Experiment Station Technical Memorandum No. 6-370, September 1953, titled "Test Data, Concrete Aggregates in Continental United States," Volume 1, Area 3, Western United States, indicates that the vicinity of the Seven Oaks Dam project has a number of sources capable of producing aggregates suitable for use in concrete construction. In accordance with EM 1110-2-2000 some of these sources were sampled and tested at the South Pacific Division Laboratory by the Los Angeles District in the Spring of 1985. The names and locations of the sources are shown on plate A-96.

11-17 Testing consisted of petrographic analysis, elementary physical tests, tests for potential reactivity of aggregates and a concrete check mix. The tests indicate that aggregates suitable for use in all aspects of concrete construction are available from local sources. The results of the laboratory tests are shown in tables A-62 through A-66.

On-Site Source

11-18 Due to the nature of materials available and the types of structures to be constructed at the site, an investigation into on-site borrow materials as sources of aggregates is being conducted. Work completed to this time includes trenching and sampling from excavated pits, in-situ gradations of materials, first stage crushing of materials, and petrography and preliminary physical testing of materials for aggregate production.

Geologic Aspects of Aggregate Sources

11-19 All aggregates from suppliers listed herein are mined from major stream beds on the alluvial plains several miles downstream from the southern margin of the San Bernardino Mountains or from streambed deposits in the vicinity of the project. The aggregates are processed from alluvium which is derived from rocks exposed in the mountain regions surrounding the streams and their tributaries. The igneous and metamorphic basement complex of the San Bernardino Mountains is the most prominent terrane in the source area, consequently most of the aggregates are composed of those rock types.

Geologic Aspects of On-Site Source

11-20 The materials considered to be potential sources of aggregate are streambed deposits of Late Quaternary age which consist mainly of cobbles and boulders as large as 5 feet in diameter, in an unconsolidated sand and gravel matrix. These deposits may extend up to 105 feet or more in thickness at the damsite and have been transported by the Santa Ana River and its tributaries which drain through two or more rock types. The primary sources are intrusive rocks of Mesozoic age, mostly quartz monzonite with some Precambrian age gneiss. This basement rock comprises much of the San Bernardino Mountains. Another minor source is the Santa Ana Sandstone formation which overlies the granitic and gneissic basement rocks in the area near the confluence of the Santa Ana River and Bear Creek. Conglomeratic portions of this unit contain schist and metavolcanic fragments as seen in the aggregate samples. A third possible minor source is the Mill Creek formation located in Warm Springs Canyon. A portion of this formation is well indurated and composed of arkosic sandstone and conglomerate with gneiss, schist, and granitic clasts. Downstream from Warm Springs Canyon and the North Branch of the San Andreas fault, the highly fractured and weathered diorite and gneiss bedrock contribute angular to subangular clasts to the streambottom deposts. The coarse aggregate was sampled from test trenches excavated in the Santa Ana River streambottom at the project site. The petrographic analysis of these samples indicate 89 percent mottled black and white granitic content with minor amounts of schist, gneiss and metavolcanics. Fine aggregate samples consist of granitics, schist, quartz fragments and feldspar fragments. The majority of the aggregate samples were fresh and hard, characteristics needed to better withstand the effects of stream transportation.

Geologic Aspects of Off-Site Sources

11-21 **LYTLE CREEK.** Both Owl Rock Company and Fourth Street Crusher aggregate sources are located in the Lytle Creek streambed 4 and 6 miles downstream from the mouth of the Lytle Creek canyon, respectively. Most of the rocks in the Lytle Creek basin are Tertiary diorite, Cretaceous granite and diorite, pre-Cretaceous Pelona schist, and various Precambrian Metamorphic rocks. The granite, granodiorite, and diorite predominate in samples from these suppliers, along with significant amounts of schist, gneiss, quartzite, and various metasedimentary and metavolcanic rocks. Minor amounts of gabbro are also present, from Mesozoic exposures west of the canyon. Fine aggregates (No. 4 and smaller particle size) produced from these two sources contains fragments of all of the above rocks, along with individual grains of hornblende, quartz, biotite, and feldspars.

11-22 **SANTA ANA RIVER.** The C.L. Pharris aggregate sources is located in the Santa Ana River stream bed approximately 5 miles downstream from the mouth of the Santa Ana River Canyon. Rock exposed in the Santa Ana River drainage area consists mainly of Mesozoic intrusives and Precambrian igneous and metamorphic rocks. Rock produced at the

C.L. Pharris plant is predominantly Mesozoic diorite, granodiorite, and gabbro, along with Precambrian schist. A small proportion of sandstone and siltstone fragments is also present, derived from scattered outcrops of Tertiary sediments. Fine aggregate consists of individual grains of feldspar, quartz, and biotite, with some fragments of granite, quartzite, schist, and metasedimentary rocks.

11-23 **SAN GORGONIO RIVER.** The Beaumont Concrete Company aggregate source is located in the San Gorgonio River floodplain about 6 miles downstream from the mouth of the San Gorgonio River Canyon. Aggregate produced by this supplier is composed primarily of Mesozoic granite, granodiorite, gabbro, and diorite. Pelona schist is also present, as well as gneiss and miscellaneous metamorphic lithologies probably derived from Precambrian outcrops. Fine aggregate from the Beaumont Concrete Company contains all of the above rock types as well as individual grains of the constituent minerals.

Trenching

11-24 Four test trenches (TT87-1 to TT87-4) were excavated in Santa Ana River Canyon to investigate the potential of processing streambed materials for use as concrete aggregates. The depths of the trenches were 15, 10, 11, and 8 feet, for TT87-1 through TT87-4, respectively. The location of the trenches is shown on plate A-97.

11-25 The test trench excavations were performed with a Caterpillar D-8H bulldozer. Some ripping was required to dislodge cobbles and boulders which were tightly interlocked. No ripping of bedrock or cemented materials occurred. The materials found in the excavation were alluvial sand, gravels and cobbles with random boulders to 40 inches in diameter. Groundwater was only encountered in one of the test trenches (TT87-2), it was noted at a depth of 9 feet. Some days later, after the water had receded, an encrustation was observed on the materials remaining in the bottom of the trench. A composite gradation of materials sampled was performed. Samples weighing 17, 15, 10, and 12 tons were excavated from the spoils pile of each pit for gradations. Sampling was performed using a Caterpillar Type 977 tracked loader. The results of the gradations are shown in plate A-98.

First Stage Crushing

11-26 After completion of in situ gradations, approximately 28 tons of representative material were removed in equal proportions from the spoil piles of TT87-3 and TT87-4. This material was loaded with a Caterpillar Type 977 tracked loader and transported to an aggregate processing plant approximately 14 miles west of the project site for primary crushing. Material transported varied from a maximum size of approximately 30 inches to the smallest sand sizes available in the excavation. Gradations of in situ materials are shown on plate A-98. The first stage crushing was performed at CALMAT'S San Bernardino Plant.

11-27 Crushing was by a 36 inch jaw crusher with an approximate capacity of 500 tons per hour. The jaws were set so that a maximum particle size of 4 inches would be produced. Preliminary screens were removed to allow the entire crushed product to be binned and stored in an aggregate silo generally used for sands. This allowed discharge of the crushed product directly into a truck for transportation. After crushing, by CALMAT, approximately 20 tons of the crushed product was shipped to the South Pacific Division Laboratory at Sausalito, California. A gradation performed on the crushed product is shown in table A-62 and on plate A-98.

Summary of Aggregate Test Data

11-28 The following information summarizes completed South Pacific Division (SPD) Laboratory testing on aggregates from sources sampled for potential construction of concrete elements for Seven Oaks Dam.

On-Site Source

11-29 The on-site source is an alluvial deposit in the immediate vicinity of the dam construction. Samples were taken from TT87-3 and TT87-4. The location of this source is shown on plate A-97. Samples were obtained in August 1987 and testing is continuing at this time. Aggregate test results are shown in table A-62 and in figure A-4.

11-30 After crushing, the SPD Laboratory performed gradations and physical property tests on the aggregates. The fine aggregate produced in the crushing operation conforms to the grading requirements of ASTM C 33 for concrete sand. Gradations determined from the samples are shown in table A-62. Only one coarse aggregate size is shown, since at this time only excavation and crushing have been completed. Additional coarse aggregate sizes will be developed and reported in feature design documents.

11-31 The rock sizes tested had a specific gravity (Sp. Gr.) of 2.63 to 2.66. The fine aggregate had a Sp. Gr. of 2.58. The aggregates had absorptions of 0.7 to 1.2 percent. The coarser sizes are less absorptive than the #4-3/4". The fine aggregate reported an absorption of 0.5 percent. Test results for abrasion loss of coarse aggregate, in accordance with ASTM C 131, showed 35 percent loss for the 2"-3" material and 21 percent loss for the 1-1/2" to 1" material. The chemical method of reactivity was performed and the results are shown in figure A-4. Test results available at this time indicate that there should be no unfavorable reactions between cements and the aggregates. Mortar bar test results will be available in the Feature Design Memorandum.

Table A-62. Physical Tests on Concrete Aggregates for:
On-Site Source, Seven Oaks Dam
(Date Tested: August 1987)

Part A: GRADATIONS IN PERCENT FINER BY WEIGHT

Sieve Size	3/8" - 4.0"	Fine Agg.
4 in.	100	
3 in.	94	
2-1/2 in.	92	
2 in.	82	
1-1/2 in.	74	
1 in.	66	
3/4 in.	59	
1/2 in.	52	
3/8 in.	48	
No. 4	39	100
No. 8	33	87
No. 16	25	65
No. 30	16	41
No. 50	7	19
No. 100	3	9
No. 200	2	4

Part B: PHYSICAL TEST RESULTS

Test Requirement	2.0" - 3.0"	3/4" - 1.5"	#4 - 3/4"	Fine Agg.
Specific Gravity	2.63	2.66	2.63	2.58
Absorption	0.7	0.7	1.2	0.5
Soft Particles	0.2	0.2	-	-

Part C: PHYSICAL TESTS ON COMBINED SAMPLES

Organic Impurities (ASTM C 40)			OK
Mortar Strength ratio @ 7 days (ASTM C 87)			108
Soundness: Magnesium Sulfate (ASTM C 88)			
Coarse Aggregate	3/4" - 1-1/2"		1.4
	No. 4 - 3/4"		2.2
Fine Aggregate			4.6
Decantation (ASTM C 117)			
Abrasion Loss, 500 rev. (ASTM C 131)			
	2" - 3"		1-1/2" - 1"
Grading Designation	A		A
Percent Loss	35		21
Reactivity, Chemical Method (ASTM C 289)			
Coarse Aggregate	Rc = 34	Sc = 21	Innocuous
Fine Aggregate	Rc = 30	Sc = 19	Innocuous

Owl Rock Products

11-32 This source is on Lytle Creek approximately 20 miles west of the project site. This source excavates alluvial sands and gravels from the Lytle Creek streambed deposits. The location of this source is shown on plate A-96. Samples of aggregates were obtained from this source in 1985 and were tested at the SPD Laboratory. Aggregate test results are shown in table A-63 and in figure A-5.

11-33 At the time of the sampling the plant was producing three rock sizes and a washed concrete sand, which were sampled for testing. The course sized materials included a 2-, 1-1/2-, and 3/8-inch topsize materials. The 2-inch material meets the Standard Specifications for Public Works Construction (SSPWC), a local specifying group made up of public agencies and suppliers, specifications size No. 2, and ASTM C 33, size No. 4. The 1-1/2-inch material meets the SSPWC specifications size No. 3 and ASTM C 33 size No. 56. The 3/8-inch material meets the SSPWC size No. 4 but does not meet any ASTM size standard. The washed concrete sand conforms to both the SSPWC and ASTM C 33 size for washed concrete sand. Gradations determined from the samples taken are shown in table A-63.

11-34 The rock sizes tested had Sp. Gr. of 2.65 to 2.67 with Sp. Gr. of 2.67 and above for aggregates greater than 3/4-inch in size. The sand had a Sp. Gr. of 2.63. The aggregates had absorptions of 1.0 to 1.9 percent for the coarse and 1.6 percent for the fines. The coarse aggregate has an abrasion loss of 24 percent when tested in accordance with ASTM C 131, using gradation A. The chemical method of reactivity of aggregates was performed and the results are shown in figure A-5. The test results indicate that there should be no unfavorable reactions between cements and the aggregates. The aggregates are generally of slightly higher quality than those found in the Los Angeles Basin, but are about average for the local San Bernardino area.

4th Street Crusher

11-35 This source is on Lytle Creek approximately 17 miles west of the project site. This source excavates alluvial sands and gravels from Lytle Creek streambed deposits near it's confluence with the Santa Ana River. The location of this source is shown on plate A-96. Samples of aggregates were obtained from this source in 1985 and were tested at the SPD Laboratory. Aggregate test results are shown in table A-64 and figures A-6 and A-9.

Table A-63. Physical Tests on Concrete Aggregates for:

OWL PRODUCTS COMPANY
Riverside Avenue at Linda Street
Rialto, California
(Date tested: January 1985)

Part A: GRADATIONS IN PERCENT FINER BY WEIGHT

Sieve Size	1.5" - 3.0"	3/4" - 1.5"	#4 - 3/8"	Fine Agg.
2 in.	100			
1-1/2 in.	96	100		
1 in.	23	98		
3/4 in.	2	74		
1/2 in.		31		
3/8 in.		14	100	100
No. 4		4	4	99
No. 8				82
No. 16				63
No. 30				41
No. 50				19
No. 100				7
No. 200				3

Part B: PHYSICAL TEST RESULTS

Test Requirement	1.5" - 3.0"	3/4" - 1.5"	#4 - 3/4"	Fine Agg.
Specific Gravity	2.67	2.67	2.65	2.63
Absorption	1.0	1.6	1.9	1.6
Soft Particles	.	0.20	0.40	.

Part C: PHYSICAL TESTS ON COMBINED SAMPLES

Organic Impurities (ASTM C 40)			OK
Mortar Strength ratio @ 7 days (ASTM C 87)			91
Soundness: Magnesium Sulfate (ASTM C 88)			
Coarse Aggregate			1.35
Fine Aggregate			3.18
Decantation (ASTM C 117)			14.50
Abrasion Loss, 500 rev. (ASTM C 131)			
Grading Designation			A
Percent Loss			24
Reactivity, Chemical Method (ASTM C 289)			
Coarse Aggregate	Rc = 36	Sc = 29	Innocuous
Fine Aggregate	Rc = 51	Sc = 43	Innocuous

11-36 At the time of sampling the plant was producing three rock sizes and a washed concrete sand, which were samples for testing. The coarse sized materials included a 2-, 1-1/2-, and 3/8-inch topsize materials. the 2-inch material meets SSPWC specifications size No. 2, and ASTM C 33, size No. 4. The 1-1/2-inch material meets the ASTM C 33 size No. 56 and barely fails the SSPWC size No. 3 on the 3/8-inch screen. The 3/8-inch material does not meet any specific size standard. The washed concrete sand conforms to both the SSPWC and ASTM C 33 size for washed concrete sand. Gradations determined from the samples taken are shown in table A-64.

11-37 The rock sizes tested had Sp. Gr. of 2.65 to 2.69 with Sp. Gr. of 2.68 and above for aggregates greater than 3/4-inch in size. The sand had a Sp. Gr. of 2.63. The aggregates had absorptions of 1.0 to 1.6 percent for the coarse and 1.2 percent for the fines. The coarse aggregate has an abrasion loss of 25 percent when tested in accordance with ASTM C 131, using gradation A. The chemical method of reactivity of aggregates was performed and the results are shown in figure A-6. The test results indicate that there should be no unfavorable reactions between cements and the aggregates. Mortar bar test results show expansion of +0.025 percent at 180 days and +0.012 percent at 335 days for high alkali cement. Expansions of +0.009 percent at 180 days and 360 days are noted for low alkali cements. Detailed results are shown in figure A-9. The aggregates are generally of slightly higher quality than those found in the Los Angeles Basin, but are about average for the local San Bernardino area.

11-38 Subsequent review of this source in December 1987 indicated that the pit was no longer able to produce coarse aggregates. The only material being produced from the pit at this time was a concrete sand. Large size materials are periodically washed into the pit as a result of stream flows. The extended drought has introduced no coarse materials into the pit, and as a consequence this source at this time does not supply coarse aggregates. If materials are deposited subsequent to this report it is anticipated that they will have properties similar to those described herein. In accordance with SPD criteria, this source and or any other source proposed as capable of producing acceptable material will be tested prior to inclusion in plans and specifications.

Table A-64. Physical Tests on Concrete Aggregates for:

4TH STREET CRUSHER
1945 W. 4TH Street on Lytle Creek
Rialto, California
(Date tested: January 1985)

Part A: GRADATIONS IN PERCENT FINER BY WEIGHT

Sieve Size	1.5" - 3.0"	3/4" - 1.5"	#4 - 3/8"	Fine Agg.
2 in.	100			
1-1/2 in.	97	100		
1 in.	27	99		
3/4 in.	2	66		
1/2 in.		22		
3/8 in.		4	100	100
No. 4			8	97
No. 8				86
No. 16				68
No. 30				41
No. 50				18
No. 100				6
No. 200				2

Part B: PHYSICAL TEST RESULTS

Test Requirement	1.5" - 3.0"	3/4" - 1.5"	#4 - 3/4"	Fine Agg.
Specific Gravity	2.69	2.68	2.65	2.63
Absorption	1.0	1.1	1.6	1.2
Soft Particles	.	1.3	0.1	.

Part C: PHYSICAL TESTS ON COMBINED SAMPLES

Organic Impurities (ASTM C 40)			OK
Mortar Strength ratio @ 7 days (ASTM C 87)			80
Soundness: Magnesium Sulfate (ASTM C 88)			
Coarse Aggregate	3/4" - 1-1/2"		1.5
	#4 - 3/4"		2.3
Fine Aggregate			12.6
Decantation (ASTM C 117)			
Abrasion Loss, 500 rev. (ASTM C 131)			
Grading Designation			A
Percent Loss			25
Reactivity, Chemical Method (ASTM C 289)			
Coarse Aggregate	Rc = 49	Sc = 20	Innocuous
Fine Aggregate	Rc = 28	Sc = 27	Innocuous

Beaumont Concrete Co.

11-39 This source is near Cabazon approximately 11 miles east of the project site. This source excavates alluvial sands and gravels. The location of this source is shown on plate A-96. Samples of aggregates were obtained from this source in 1985 and were tested at the SPD Laboratory. Aggregate test results are shown in table A-65 and figures A-7 and A-10.

11-40 At the time of sampling the plant was producing three rock sizes and a washed concrete sand, which were samples for testing. The course sized materials included a 2-, 1-1/2-, and 3/8-inch topsize materials. The 2-inch material meets the Standard Specifications for Public Works Construction (SSPWC), a local specifying group made up of public agencies and suppliers, specifications size No. 2, and ASTM C 33, size No. 4. The 1-1/2-inch material meets the SSPWC specifications size No. 3 and ASTM C 33 size No. 56. The 3/8-inch material does not meet any size standard. The washed concrete sand conforms to both the SSPWC and ASTM C 33 size for washed concrete sand. Gradations determined from the samples taken are shown in table A-65.

11-41 The rock sizes tested had Sp. Gr. of 2.63 to 2.67 with Sp. Gr. of 2.68 and above for aggregates greater than 3/4-inch in size. The sand had a Sp. Gr. of 2.66. The aggregates had absorptions of 1.0 to 2.7 percent for the coarse and 1.2 percent for the fines. The coarse aggregate has an abrasion loss of 39.1 percent when tested in accordance with ASTM C 131, using gradation A. This result is slightly higher than desired but conforms to ASTM and SSPWC requirements. The chemical method of reactivity of aggregates was performed and the results are shown in figure A-7. Mortar bar test results show expansion of +0.044 percent at 180 days and +0.040 percent at 335 days for high alkali cements. A peak of +0.046 percent at 225 days. Expansions of +0.012 percent at 180 days and +0.014 percent at 335 days with a peak of +0.016 percent at 55 days were noted for low alkali cements. Although the values are below limits for expansion +0.05 percent at 180 days and +0.10 percent at 360 days, these reaction results are much higher than most results in the Los Angeles District. Use of these aggregates may require special requirements or may not be suitable for use in the types of construction anticipated.

Table A-65. Physical Tests on Concrete Aggregates for:

BEAUMONT CONCRETE CO.
Cabazon Pit
San Gorgonio River at Apache Trail
Cabazon, California
(Date tested: January 1985)

Part A: GRADATIONS IN PERCENT FINER BY WEIGHT

Sieve Size	1.5" - 3.0"	3/4" - 1.5"	#4 - 3/4"	Fine Agg.
2 in.	100			
1-1/2 in.	94	100		
1 in.	21	99		
3/4 in.	2	74	100	
1/2 in.		33	99	
3/8 in.		11	56	100
No. 4		2		99
No. 8				86
No. 16				66
No. 30				43
No. 50				19
No. 100				6
No. 200				2

Part B: PHYSICAL TEST RESULTS

Test Requirement	1.5" - 3.0"	3/4" - 1.5"	#4 - 3/4"	Fine Agg.
Specific Gravity	2.66	2.67	2.63	2.66
Absorption	1.0	1.3	2.7	1.2
Soft Particles	0.0	0.0	0.0	.

Part C: PHYSICAL TESTS ON COMBINED SAMPLES

Organic Impurities (ASTM C 40)			OK
Mortar Strength ratio @ 7 days (ASTM C 87)			56
Soundness: Magnesium Sulfate (ASTM C 88)			
Coarse Aggregate	3/4" - 1-1/2"		1.9
	#4 - 3/4"		2.5
Fine Aggregate			16.1
Decantation (ASTM C 117)			
Abrasion Loss, 500 rev. (ASTM C 131)			
Grading Designation			A
Percent Loss			39.1
Reactivity, Chemical Method (ASTM C 289)			
Coarse Aggregate	Rc = 36	Sc = 29	Innocuous
Fine Aggregate	Rc = 51	Sc = 43	Innocuous

C.L. Pharris

11-42 This source is on the Santa Ana River approximately 6 miles west of the project site. This source excavates alluvial sands and gravels from Santa Ana River streambed deposits. The location of this source is shown on plate A-96. Samples of aggregates were obtained from this source in 1985 and were tested at the SPD Laboratory. Aggregate test results are shown in table A-66 and figures A-8 and A-11.

11-43 At the time of sampling the plant was producing three rock sizes and a washed concrete sand, which were sampled for testing. The coarse sized materials included a 2-, 1-1/2-, and 3/8-inch topsize materials. The 2-inch material meets the SSPWC specifications size No. 2, and ASTM C 33, size No. 4. The 1-1/2-inch material failed the SSPWC specifications for size No. 3 on the 3/8-inch screen, but meets ASTM C 33 size No. 56. The 3/8-inch material does not meet any size standard. The washed concrete sand failed both the SSPWC and ASTM C 33 size for washed concrete sand due to the presence of excess fines. Gradations determined from the samples taken are shown in table A-66.

11-44 The rock sizes tested had Sp. Gr. of 2.62 to 2.66 with Sp. Gr. of 2.65 and above for aggregates greater than 3/4-inch in size. The sand had a Sp. Gr. of 2.63. The aggregates had absorptions of 0.8 to 1.7 percent for the coarse and 1.1 percent for the fines. The coarse aggregate has an abrasion loss of 32.7 percent when tested in accordance with ASTM C-131, using gradation A. The chemical method of reactivity of aggregates was performed and the results are shown in figure A-8. The test results indicate that there should be no unfavorable reactions between cements and the aggregates. Mortar bar test results show expansions of +0.025 percent at 180 days and +0.015 percent at 335 days with a peak of +0.030 percent at 235 days for high alkali cement. Expansions of +0.021 percent at 180 days and +0.013 percent at 335 days are noted for low alkali cements. Detailed results are shown in figure A-11. The aggregates are generally of slightly higher quality than those found in the Los Angeles Basin, but are about average for the local San Bernardino area.

Table A-66. Physical Tests on Concrete Aggregates for:

C.L. PHARRIS
E. of Norton AFB
San Bernardino, California
(Date tested: January 1985)

Part A: GRADATIONS IN PERCENT FINER BY WEIGHT

Sieve Size	1.5" - 3.0"	3/4" - 1.5"	#4 - 3/4"	Fine Agg.
2 in.	100			
1-1/2 in.	95	100		
1 in.	18	99		
3/4 in.	1	70	100	

Table A-66. (Continued)

Part A: GRADATIONS IN PERCENT FINER BY WEIGHT

Sieve Size	1.5" - 3.0"	3/4" - 1.5"	#4 - 3/4"	Fine Agg.
1/2 in.	1	23	99	
3/8 in.		3	10	100
No. 4		1		98
No. 8				88
No. 16				70
No. 30				41
No. 50				17
No. 100				6
No. 200				6

Part B: PHYSICAL TEST RESULTS

Test Requirement	1.5" - 3.0"	3/4" - 1.5"	#4 - 3/4"	Fine Agg.
Specific Gravity	2.66	2.65	2.62	2.63
Absorption	0.8	0.9	1.7	1.1
Soft Particles	0.0	1.5	0.3	.

Part C: PHYSICAL TESTS ON COMBINED SAMPLES

Organic Impurities (ASTM C 40)			OK
Mortar Strength ratio @ 7 days (ASTM C 87)			77
Soundness: Magnesium Sulfate (ASTM C 88)			
Coarse Aggregate	3/4" - 1-1/2"		0.5
	#4 - 3/4"		1.9
Fine Aggregate			10.4
Decantation (ASTM C 117)			
Abrasion Loss, 500 rev. (ASTM C 131)			
Grading Designation			A
Percent Loss			32.7
Reactivity, Chemical Method (ASTM C 289)			
Coarse Aggregate	Rc = 41	Sc = 26	Innocuous
Fine Aggregate	Rc = 51	Sc = 24	Innocuous

WATER

11-45 Water for concrete and embankment construction is available in sufficient quantities from the Foothill Pipeline located at the mouth of Santa Ana River Canyon. Arrangements for purchase of the water from the State of California would be made by the Corps instead of the

Contractors because of the number of construction contracts to be issued in a relatively short time, and because of the complexity of the water exchange agreements between the various water interests in the vicinity.

11-46 An alternative would be to extract water from the alluvial deposits in the streambed in the immediate vicinity of the Dam. Observations made during geotechnical explorations indicated the presence of some mineral encrustation left after groundwater had evaporated from one of the test trenches. In order to determine the potential affects of this on concrete a study to determine the mineral characteristics of the water will be performed during feature design studies. Water available from the streambed and water available from commercial sources must be analyzed.

ADMIXTURES

11-47 Admixtures for concrete construction, based on the preliminary design information available at this time, will probably be limited to regular water reducers, retarders and air entraining admixtures. These admixtures would be specified to conform to the requirements of ASTM C 494. No need has been determined for the use of high range water reducers or superplasticizers at this time. Detailed information concerning the use of admixtures will be developed in the Feature Design Memorandum.

RECOMMENDATIONS

11-48 At this point in time, recommendations can be divided into two parts. First, specific laboratory studies to determine material and mix design properties; and second, specification requirements for construction which can be made based on design and material data available at this time.

Laboratory Studies

11-49 Laboratory studies to determine the basic physical parameters of aggregate sources have been completed. The next phase of laboratory studies will be to determine mixture proportions for the various classes of concrete required and physical measurement of the mixture properties will be determined. Laboratory studies will be performed at the South Pacific Division (SPD) Laboratory, the North Pacific Division (NPD) Laboratory, and the Waterways Experiment Station (WES).

11-50 Studies to be performed at the SPD Laboratory, in addition to the basic physical parameter determinations already completed, will be aggregate screening and production studies, mixture proportioning and a class 'C' flyash suitability determination. Screening and production studies will be used to manufacture various size groups of aggregates to prepare mixture proportions for the various concretes required.

11-51 Class 'C' ashes will be analyzed to determine their suitability for use in the types of structures to be constructed. In December 1987, studies were started to determine suitable replacement rates of C ash in

concrete mixtures. Once this rate is determined, full scale mixes will be prepared for further testing. Long term testing by mortar bar and chemical soundness tests will simultaneously be performed.

11-52 Mixture design studies will be performed to develop baseline data for concrete mixture proportions for various structural elements. Mixtures will be developed for the foundation mat, inlet tower, tunnel lining, outlet invert, and outlet walls. Additionally, mixes for use in shotcrete applications will be developed.

11-53 The NPD Laboratory will perform coefficient of thermal expansion and thermal diffusivity studies on 2-foot cubes of concrete based on mixtures proposed for use in the foundation mat of the inlet tower. The cubes will be fabricated by the SPD Laboratory and then shipped to NPD. The SPD Laboratory does not have the necessary equipment for performing these tests.

11-54 WES will perform tests on cements and pozzolans to assist in developing material performance criteria. WES has already completed a partial analysis of a Class 'C' flyash in support of the 'C' ash investigations described above.

Mix Designs

11-55 Mix designs will be developed at the SPD Laboratory for the following structural elements.

Table A-67. Mix Design Requirements for Structural Elements.

Concrete Class	Maximum Aggregate Size	Compressive Strength Requirements (psi) (@ age)
Foundation Mat	4"	4000 @ 365 days
Inlet Tower Walls	4"	3000 @ 56 days
	2"	3000 @ 56 days
Tunnel Lining	1-1/2"	3000 @ 28 days
Outlet Walls	3/4"	3000 @ 28 days
Invert	1-1/2"	3000 @ 28 days
Shotcrete	3/8"	4000 @ 7 days

Specification Requirements

11-56 The following summarizes preliminary assumptions based on the partial lab work completed.

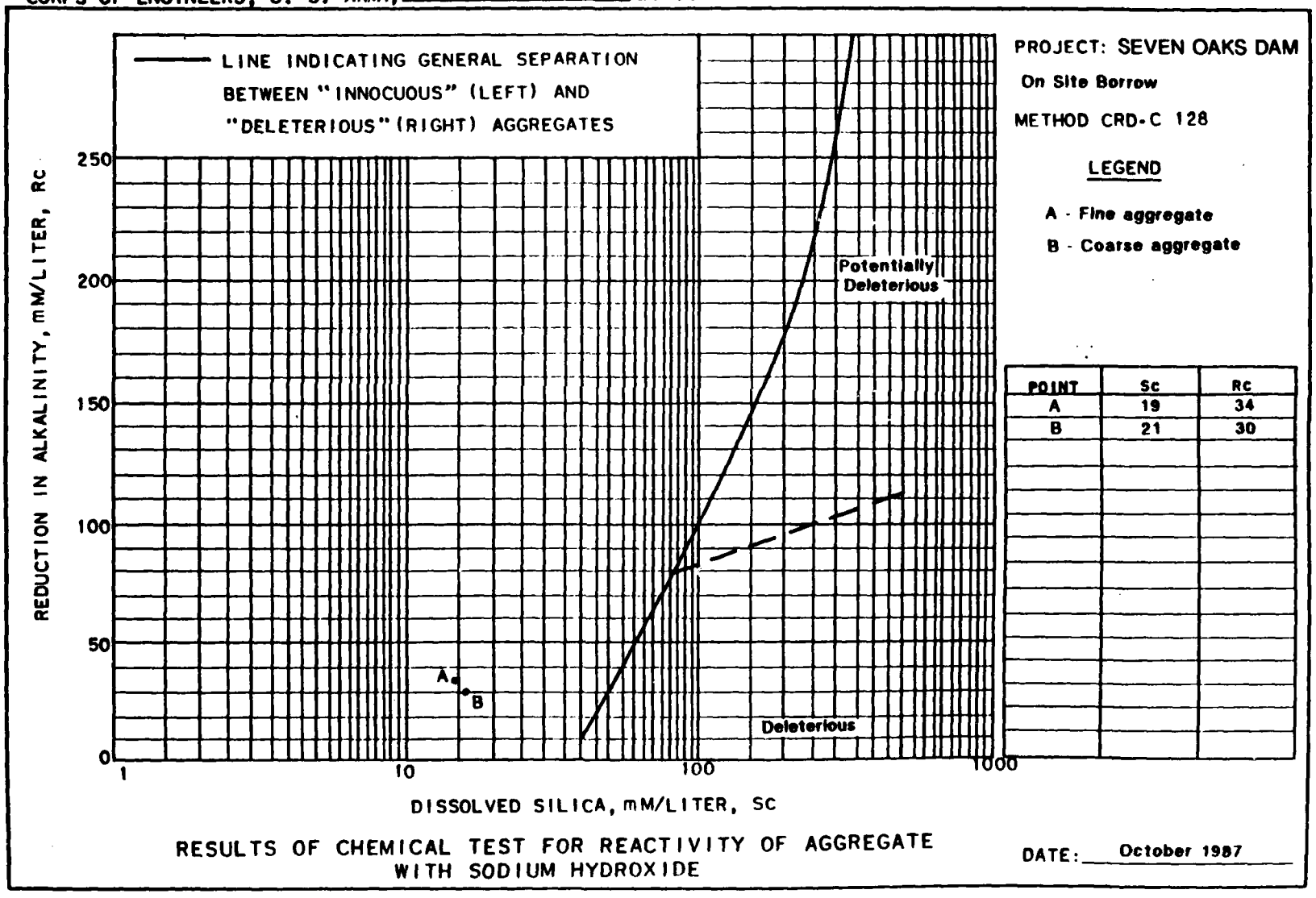
11-57 Laboratory studies will be performed in order to determine requirements for cements, pozzolans, and water. No special or unusual requirements are anticipated.

11-58 Admixtures will most likely be limited to water reducers and air entraining admixtures.

11-59 Based on preliminary data, suitable aggregates could be obtained from either on-site or existing aggregate pits in the region.

11-60 Instrumentation of the concrete will be proposed. Present plans are to insert temperature and strain gauges in the foundation mat and strain gauges in the walls of the inlet tower. This instrumentation would be for short term modeling and analysis at the structure and would not be part of a permanent instrumentation plan. A permanent plan will be developed and detailed in the Feature Design Memorandum. Specific details of placing of concrete will be developed in the Feature Design Memorandum. Based on present plans and information, placing of th foundation mat would be by bucket or belt only. Placing of the tower concrete will probably be by bucket only. Pump placement would only be allowed in the tunnel lining and in the outlet invert and walls.

CORPS OF ENGINEERS, U. S. ARMY, South Pacific DIVISION.



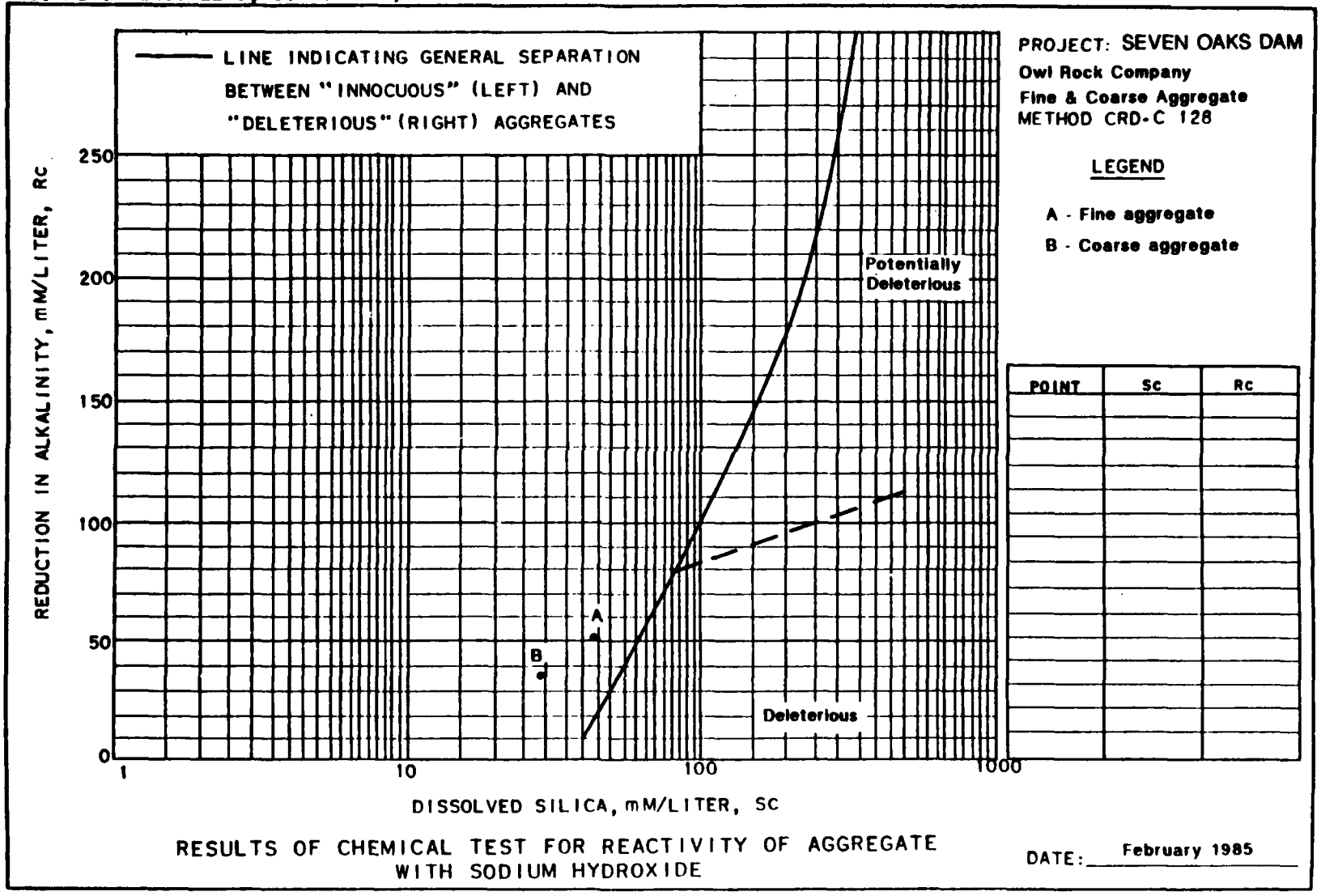
RESULTS OF CHEMICAL TEST FOR REACTIVITY OF AGGREGATE WITH SODIUM HYDROXIDE

DATE: October 1987

WES FORM 896
DEC. '55

FIGURE A-4

CORPS OF ENGINEERS, U. S. ARMY, South Pacific DIVISION.



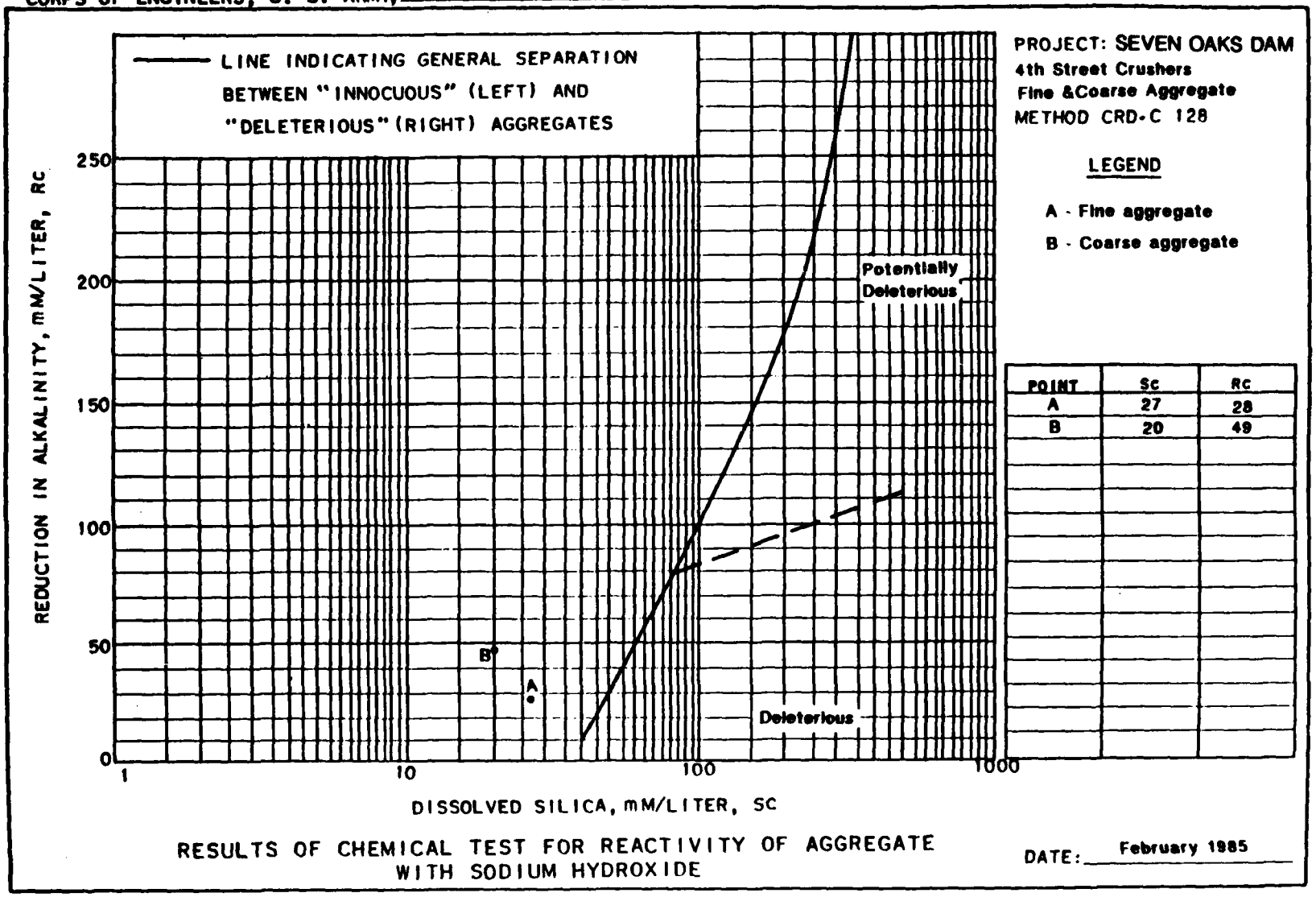
RESULTS OF CHEMICAL TEST FOR REACTIVITY OF AGGREGATE WITH SODIUM HYDROXIDE

DATE: February 1985

WES FORM
DEC. '55 896

FIGURE A-5

CORPS OF ENGINEERS, U. S. ARMY, South Pacific DIVISION.



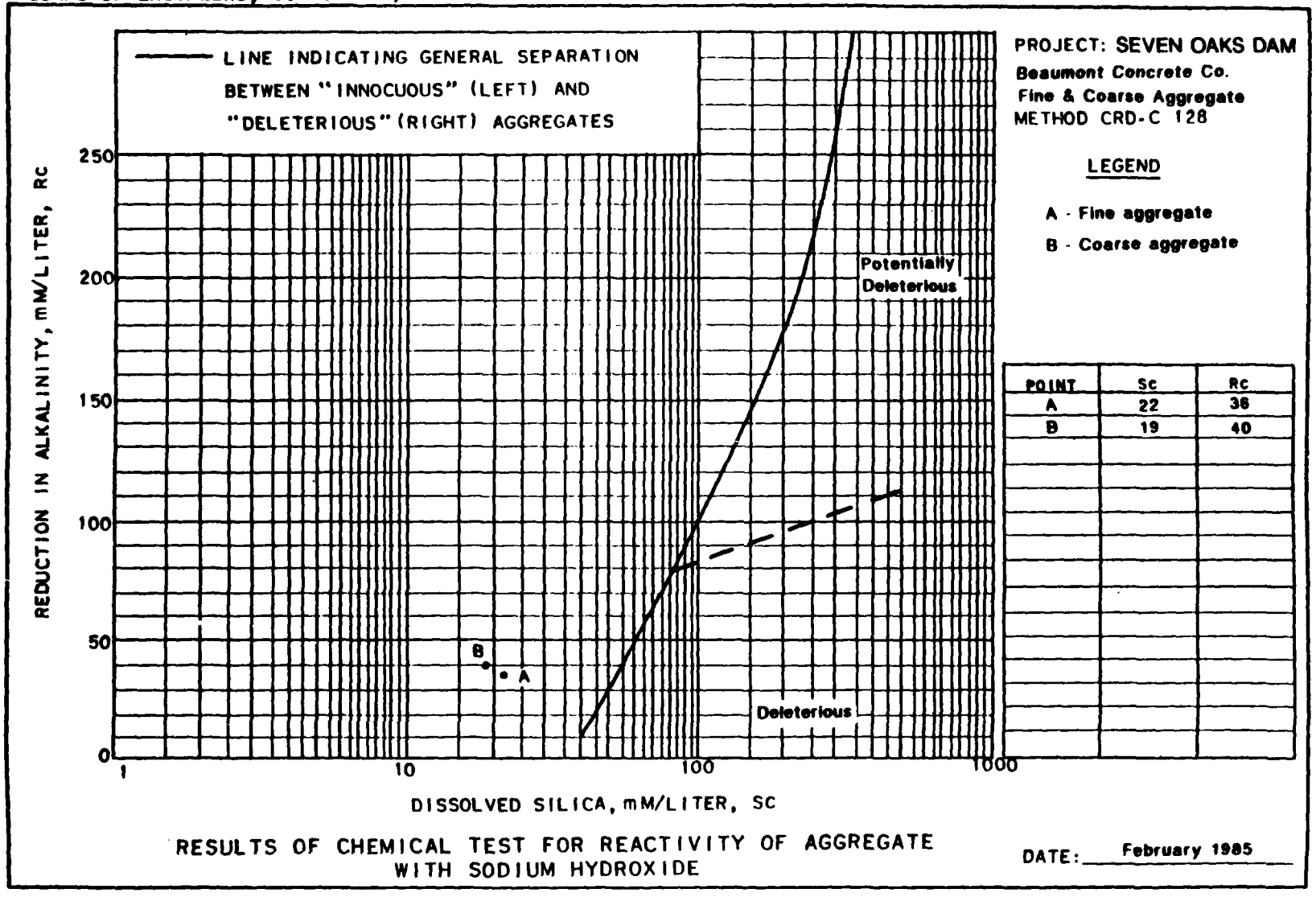
RESULTS OF CHEMICAL TEST FOR REACTIVITY OF AGGREGATE WITH SODIUM HYDROXIDE

DATE: February 1985

WES FORM DEC. '55 896

FIGURE A-6

CORPS OF ENGINEERS, U. S. ARMY, South Pacific DIVISION.



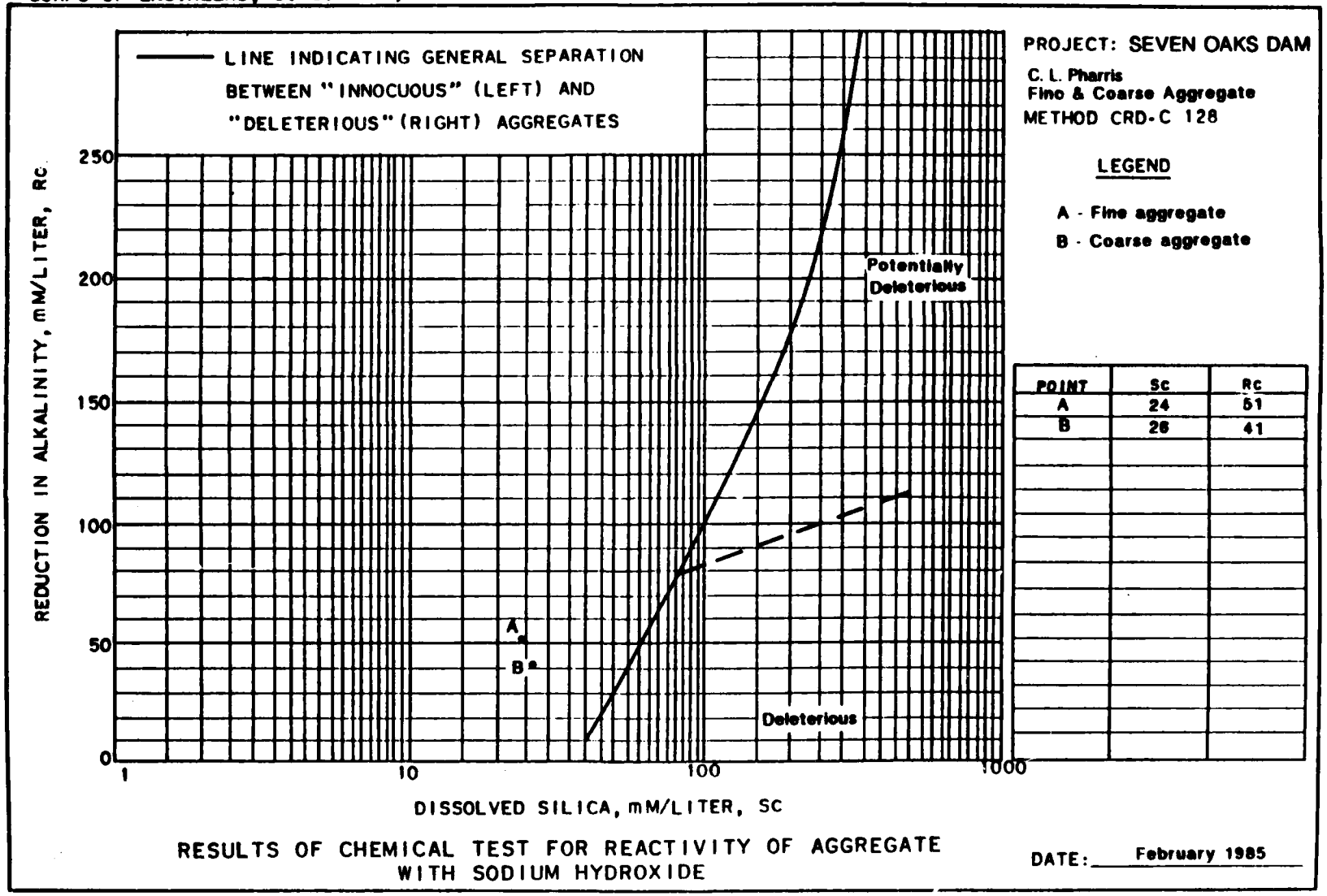
RESULTS OF CHEMICAL TEST FOR REACTIVITY OF AGGREGATE WITH SODIUM HYDROXIDE

DATE: February 1985

WES FORM DEC. '55 896

FIGURE A-7

CORPS OF ENGINEERS, U. S. ARMY, South Pacific DIVISION.

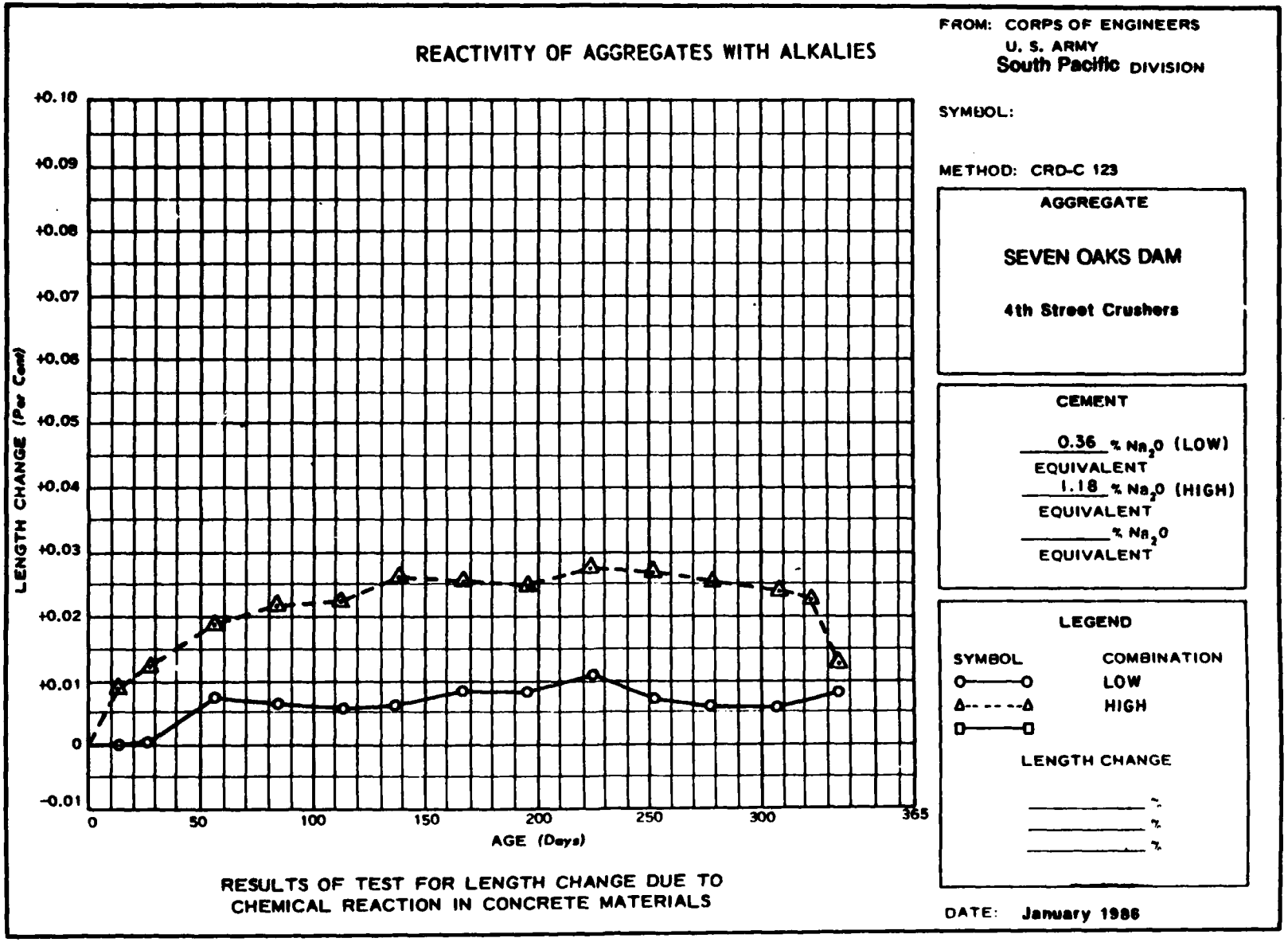


RESULTS OF CHEMICAL TEST FOR REACTIVITY OF AGGREGATE WITH SODIUM HYDROXIDE

DATE: February 1985

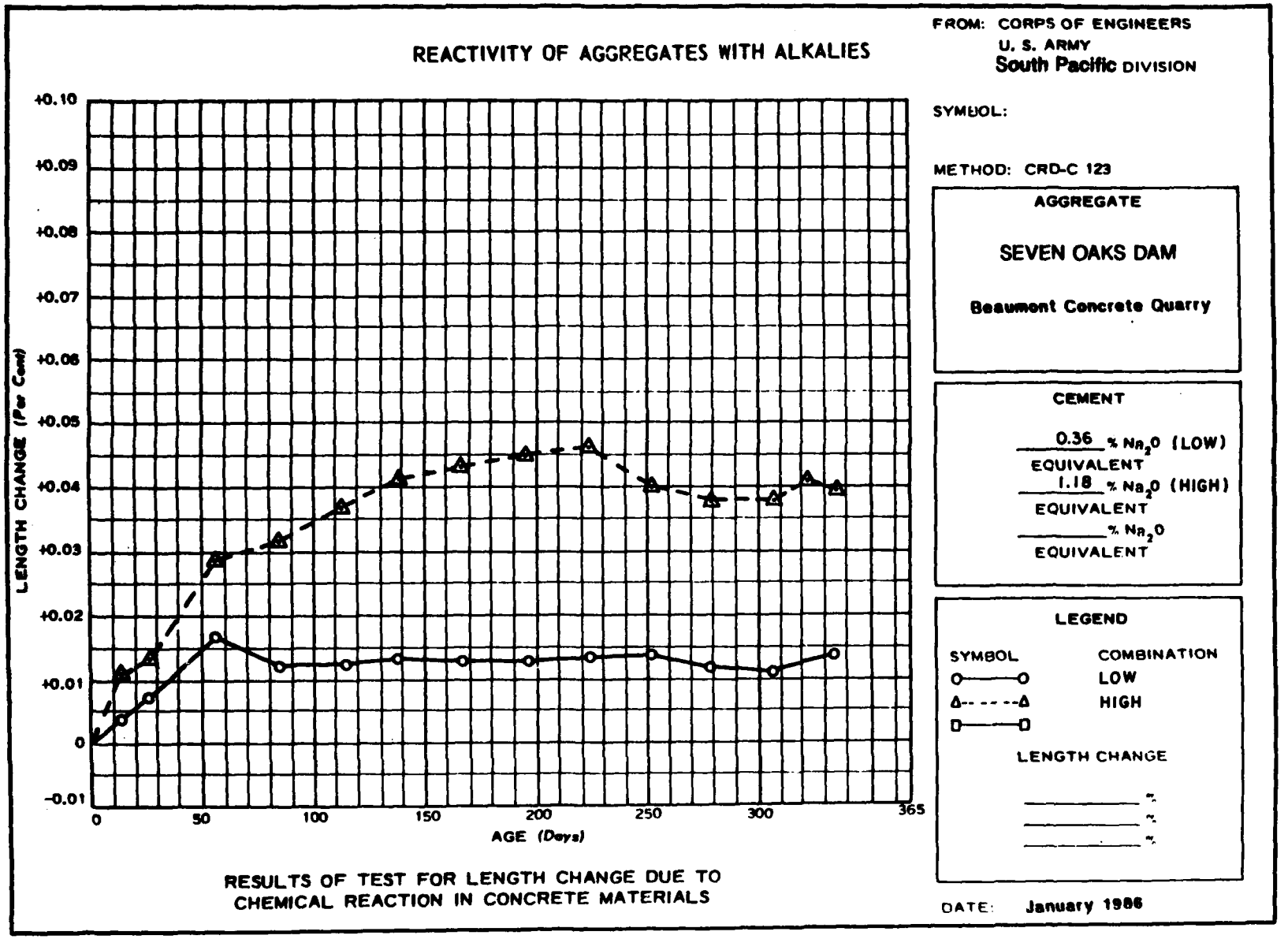
WES FORM
DEC. '55 896

FIGURE A-8



WES FORM 895
DEC 1955

FIGURE A-9



WES FORM 895
DEC 1955

FIGURE A-10

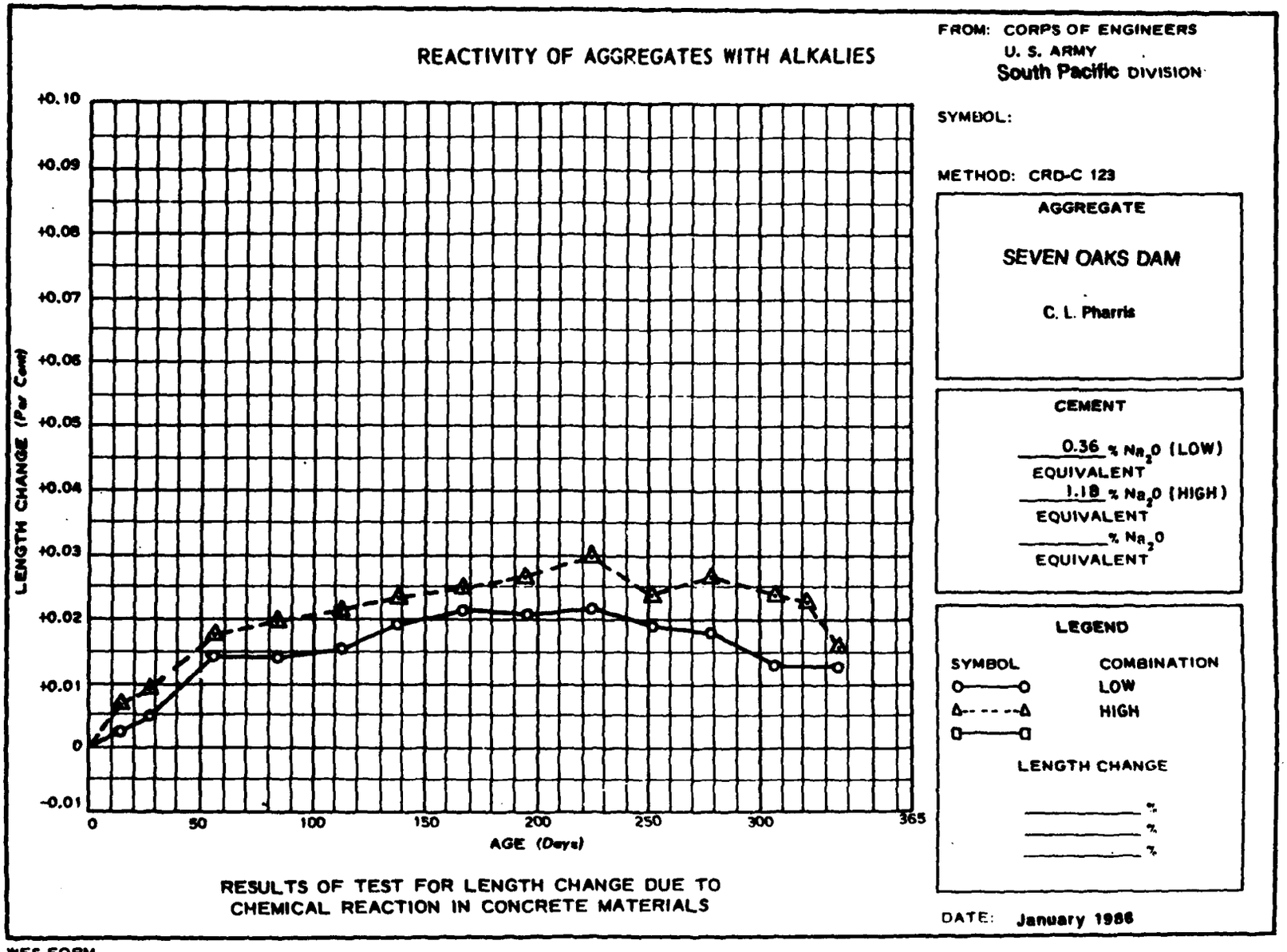


FIGURE A-11

WES FORM 895
DEC 1955

XII. CONSTRUCTION PROCEDURES AND REQUIREMENTS

Diversion and Control of Water

12-01 The Santa Ana River is a relatively small but perennial river under normal conditions. Investigations have shown that groundwater is usually within 10 to 20 feet of the surface at the proposed damsite. Two studies have both calculated that approximately 3 ft^3/s is flowing through the streambed aquifer. Although design of diversion and control facilities will probably be the responsibility of the Contractor, for estimating purposes a dewatering program to control the flow of groundwater through the site during excavation and preparation of the foundation, and a cofferdam design to divert surface flows into the outlet tunnel have been developed and are described below.

SLURRY TRENCH CUTOFF

12-02 An earth-backfill slurry trench cutoff would be constructed across the canyon just upstream of the embankment toe by excavating a trench with a clamshell through the streambed alluvium to the bedrock. The width of the trench would be at least 6 feet in order to remove the boulders that are anticipated to be encountered. The trench would be kept open with a slurry suspension of bentonite in water and backfilled with a combination of slurry and impervious material from the Mill Creek core borrow area. A 5 percent concentration of dry powdered bentonite in water would be used to provide an impervious filter cake on the walls of the trench and stability through hydrostatic pressure against the trench walls. The density of the slurry and suspended noncolloidal particles from excavation was estimated to be 75 lb/ft^3 for stabliity calculations. Density would be monitored during construction. Additional stability would be provided if necessary by building berms adjacent to the trench and raising the elevation of the top of the slurry and thereby increasing the hydrostatic force. The maximum density of the slurry during backfill operations would be specified to be at least 15 lb/ft^3 less than the density of the blended backfill material. Impervious borrow material would be mixed with the bentonite slurry with a dozer in a flat area next to the trench to achieve a

homogeneous backfill material with suitable properties. Construction control would be achieved with gradation and concrete slump tests on each batch of blended backfill prior to placement in the trench.

COFFERDAM

12-03 The cofferdam would be constructed with alluvial materials from the foundation excavation or upstream pervious borrow area. The materials would be processed through a grizzley to produce minus 12-inch and 12 to 30-inch fill materials corresponding to zone 3 and zone 5 embankment materials, respectively. After excavation of the top 20 feet of the streambed alluvium and proof rolling of the foundation, minus 12-inch material would be used to construct the majority of the cofferdam in order to assure a dense structure that can be well compacted, inspected and tested. A thick zone of 12 to 30-inch rock would comprise the crest and the downstream slope of the cofferdam to provide erosion resistance and to minimize the potential of sudden failure in the event of overtopping by flood flows. To reduce through seepage of impounded water into the foundation excavation, an impervious blanket, with filter and cover layers, would be placed on the upstream face of the cofferdam and connected to the slurry trench cutoff. After raising of the embankment to the elevation of the top of the cofferdam, the impervious layer would be removed and replaced with rock shell materials. A section illustrating the cross section of the cofferdam and its location within the embankment is shown on plate A-80.

DEWATERING WELLS

12-04 Dewatering wells will be required downstream of the foundation excavation trench to collect groundwater which would otherwise drain back into the excavation area. After initial dewatering, only water passed through the outlet tunnel during diversion would need to be controlled.

Outlet Works

PORTALS

12-05 Details of the methods and procedures for constructing the outlet works are presented in Volume 1, Appendix B and supplement, Seven Oaks Dam Outlet Works, prepared by the Portland District. An overview of the construction is given in the following paragraphs.

Excavation

12-06 Prior to actual excavation of both portals, all loose rock and overburden which could impact above the portal areas would be removed. At the inlet, the removal would be minimal and only involve scaling of scattered loose surface rock. The present natural slope is stable and it will be disturbed as little as possible. At the outlet, removal would involve all colluvium above the portal and possibly to the top of

the channel walls. Rock excavation of the portal faces and side walls would be accomplished by systematic blasting in lifts. All blasting will be closely controlled so that open fracturing is not induced into the final slopes. Pre-splitting will likely be used along the final cut lines with buffer zones in front. It is anticipated that the lift thickness will be no more than 10 feet such that the newly exposed slope can be easily secured. Should zones of softer or very broken rock be encountered which are not suitable for blasting, the material will be removed by ripping or other similar mechanical means. Blasting for the intake and outlet structures subgrade will also be carefully controlled to avoid damage to the floor. It is expected that little dewatering will be required in the foundation excavations since the bedrock appears to be tight at both portals.

Reinforcement

12-07 Contemporaneous with the excavation, each lift of newly exposed final cut surface in rock will be secured as soon as possible to inhibit stress relief. The reinforcement will consist of a pattern of grouted rock bolts set to varying depths. Wire mesh will be installed over the full cut to restrict surface rock falls.

CHANNELS AND ENERGY DISSIPATOR

12-08 An entrance channel and a diversion dike will be required to divert surface flows into the outlet works tunnel during construction of the embankment. Just beyond the downstream end of the tunnel, an energy dissipator will be excavated in the streambed.

Excavation

12-09 As the portal excavation progresses downward, it will merge into the channel excavation. Cofferdams will be constructed across the ends of both channels to protect them from floods during construction. In the energy dissipator, excavation and placement of the slope protection would be done during the summer when streamflows are low. Most of the two channels, and all of the energy dissipator are in loose cohesionless material which can be excavated by common mechanical means. The bedrock encountered near both portals will be excavated by controlled blasting as will be the portals themselves. No slope protection or reinforcement will be required in any of the channel cuts since the slopes are temporary. Dewatering is expected to be minimal in the channels, but can be significant in the energy dissipator.

Disposition of Materials

12-10 Except for cleared and grubbed surficial materials, all of the alluvium produced by excavation of the entrance channel and energy dissipator can be processed through a grizzley and used in zones 3 and 5 in the embankment. The diversion dike can be constructed with materials excavated from the entrance channel without processing since it is a temporary structure.

TUNNEL AND SHAFT

Excavation

12-11 The first step in the tunnel excavation will be to prepare both portals for "turning under". Following installation of the rock bolt arch, an "umbrella" will be added below the arch and a few inches above the outside excavation line. The umbrella would consist of 3-inch diameter holes spaced 18 inches apart, about 20 feet deep and extending from spring line to spring line over the crown. Black pipe will then be inserted in the holes with several feet protruding to allow installation of a support rib. As the tunnel excavation starts, the rock will break to the umbrella, but not beyond it, thereby preserving the corner. Because the tunnel is relatively short, excavation will likely be from only one heading, starting at the outlet. This direction will protect the work from floodflows and permit down gradient drainage should groundwater be encountered.

12-12 The tunnel will be driven by the top heading and bench method in which the upper half, above the spring line, will be excavated first and support installed followed by excavation and support of the lower half. This method will permit the blocky rock to be more easily secured. All excavation will be by systematic drilling and blasting probably in headings 3 to 5 feet in length. This length will allow one rib at a time to be set in order to secure the opening as soon as possible. Should mid-tunnel control be selected, the shaft excavation would be similar to that for the tunnel, except that the heading would proceed full face. At the tunnel/shaft junction, excavation would proceed very carefully in order to maintain stability of the chamber walls. In both structures, hole patterns and dymamite load factors will be adjusted depending on the rock conditions encountered. At this time, the load factor is estimated to be about one pound of dynamite per cubic yard of rock excavated.

Support

12-13 Pending FDM investigations on the rock conditions along the outlet alinement, the initial tunnel support is planned to be U-shaped steel ribs spaced 3 to 5 feet with continuous blocking and lagging. Special support design may be required at shear crossings. Further support will be provided by the concrete lining. Steel reinforcement will be added to the lining in the upstream reaches for possible additional hydrostataic loading. No support is anticipated for the floor and much of the walls below the spring line. It is anticipated that ring beams will be used with lagging in the shaft. Support in the gate chamber excavation, however, would be more exotic and will be developed during the Outlet Works FDM.

Grouting

12-14 Following adequate curing of the concrete lining, any remaining voids in the tunnel crown and gate chamber between the lining and the rock will be backfilled with grout. Sets of pipes will be installed on

10-foot spacing with additional pipes in areas of large overbreak. The pipe will likely be installed with placement of the lining and capped to prevent being filled with concrete. Grouting would progress from the outlet upgradient to reduce grout travel as much as possible. Since the grout will consist of a heavily sanded cement and water mix, excessive travel would promote segregation. During grouting, pressures will be closely monitored to prevent rupturing the lining. Consolidation grout rings planned at the inlet and at the dam centerline would be constructed following completion of the contact grouting. A neat grout mix of cement and water would be used and additional holes would be added should open fractures be encountered.

Disposition of Materials

12-15 The rock excavated from the portals, shaft and tunnel will mostly break to less than 12-inch size and it is anticipated that materials produced would be stockpiled downstream of the dam and used in the downstream transition portion of the embankment. If unsuitable for use in the embankment, they would be used for road fill or wasted in one of the designated disposal areas.

Foundation Preparation

ABUTMENT EXCAVATION

12-16 Because of the volume of overburden to be removed from the abutments, especially the left abutment, abutment excavation will be made under a separate early contract, so that construction of the embankment will not be delayed after completion of the Embankment FDM. In addition, an early construction start excavation contract at the abutments will include construction of access roads which will be used for surface cleaning equipment and for subsequent exploratory drilling during the Embankment FDM study phase.

Right Abutment

12-17 The entire area under the footprint of the dam at the right abutment will require stripping of overburden and vegetation. Before excavation can begin, an access road from Alder Creek Road to the top of the abutment must be constructed to allow for maintenance and repair of the earthmoving equipment. The average thickness of the residual soil, colluvium, and terrace deposits, as depicted on plates A-4 and A-5, is estimated to be 5 feet. In addition, under the core and adjacent filter of the embankment, a minimum of 5 feet of surficial bedrock will be excavated by mechanical means. The bulk of right abutment excavation, however, will be as the result of abutment shaping and access requirements. The general criteria for abutment shaping will require that 15-foot wide exploration access roads will be constructed during the abutment excavation to facilitate subsequent exploratory drilling, detailed cleaning and foundation grouting of the abutment. The access roads will be 15 percent grade or flatter and will cross the dam

centerline every 100 feet in elevation. Where the abutment surface will eventually be in contact with embankment core material, the cut slopes in the bedrock will not be steeper than 1V on 1H. In all other areas the cut slopes will not be steeper than 1V on 0.75H. See plate A-103 and the discussion of the abutment excavation contract in section XIII of this appendix for details and limits of the excavation. To reduce differential settlement, minor reshaping of abrupt slope changes at the outside edges of access roads will be accomplished just ahead of placement as the embankment is being raised.

Left Abutment

12-18 The left abutment will be stripped of all vegetation and overburden prior to initiation of streambed excavation. The surficial residual soil along with the vegetation will be stockpiled for resurfacing of borrow areas or hauled to a waste disposal site. The thick mass of colluvium and debris flow material on the lower slopes under the central portion of the dam is, however, extensive and uniform enough to be used to construct access or haul roads or stockpiled for use in the embankment.

12-19 Before abutment excavation can begin, the access road from Warm Springs Canyon via the spillway to the top of the abutment must be constructed to allow for maintenance and repair of the dozers and other earthmoving equipment. Unlike the right abutment, the existing slopes at the left abutment, especially under the central portion of the dam, are only moderately steep and should not require significant shaping. A minimum of 5 feet of foundation rock will be removed under the core and filter zone of the dam. A 15-foot wide exploration access road will be constructed within the dam footprint from the top down as the abutment is excavated to allow subsequent access for drilling, cleaning and grouting equipment on the abutment. Cut slopes for the access road will not be steeper than 1V on 1H in core contact areas and 1V on 0.75H in all other areas. Localized blasting may be required during access road construction.

STREAMBED FOUNDATION EXCAVATION

12-20 The foundation excavation for the embankment consists of removing the top 20 feet of the streambed alluvium over the entire embankment contact area, removing the streambed alluvium down to bedrock within the limits shown on plate A-80, and removing a minimum of 5 feet of bedrock within the core and filter zone contact area. The thickness of bedrock actually removed will probably be dictated by the shaping of the irregular surface and is estimated to be approximately 10 feet. The base width of the excavation to bedrock will be approximately twice the height from bedrock to the crest of the dam. Excavation can be accomplished with conventional heavy construction equipment, using dozers to loosen the materials and large loaders to place them into trucks for transportation out of the excavation trench. The excavated alluvium will be stockpiled for use later in the embankment. Because of the large quantity of oversize rock, the materials would have to be

processed through a grizzley to produce suitable filter or upstream pervious shell materials. The oversize material produced by this operation will be placed in the outer oversize shell zones of the embankment. Excavated rock will be wasted.

BEDROCK SURFACE CLEANING

12-21 Upon completion of abutment excavations, including construction of exploration access roads, the entire core and filter contact area will be blown clean. A survey grid will then be established to allow detailed geologic mapping of the cleaned area. The detailed geologic map will be used to plan the subsequent subsurface exploration and curtain grouting programs. The same cleaning procedure will be followed in the streambed foundation area after excavation to bedrock.

CURTAIN GROUTING

12-22 Immediately after surveying and detailed geologic mapping, curtain grouting will commence in the streambed excavation and proceed up both abutments to the maximum flood pool elevation. Subsurface explorations conducted subsequent to the Phase I Supplemental GDM confirmed the low permeability of the bedrock foundation. Out of 94 individual pressure tests in both the gneiss and diorite, 66 tests experienced water takes less than 0.1 ft^3/min. Therefore, approximately 70 percent would have been deferred from grouting by applying commonly used criteria to define grout-take refusal. Although very little exploration has been conducted on the abutments, and none at higher elevations, it is believed that rockmass permeabilities at the abutments will be similarly low. Therefore, the curtain grouting program described in the Phase I document has been modified only slightly. It is proposed to drill and grout a single-line curtain below the core centerline on 10-foot centers. The curtain will be 50-feet deep with two 25-foot stages. It is estimated that 40 percent of the stages will be deferred from grouting and that every eighth hole will require split spacing. In addition and subsequent to drilling and grouting of the 50-foot deep curtain, every other hole will be deepened to 1/2 the maximum flood pool depth. Because of the expected good drillability of the rock and low overall rockmass permeability, the holes will be stop-grouted with 50-foot stops. Primary spacing for this deeper curtain will be 20 feet. It is estimated that the average grout take per foot of hole drilled for the entire grouting program will be 0.1 sack/ft. In the embankment FDM, computer assisted grouting will be addressed and its applicability at the Seven Oaks damsite evaluated.

BEDROCK SURFACE TREATMENT

12-23 As soon as the curtain grouting is completed in the streambed excavation and immediately before placing embankment material, the rock surface under the core and filter zones will be recleaned to remove all loose rock debris and surface contamination resulting from the curtain grouting operations. Open joints and fractures will be backfilled with slurry grout. Localized surface irregularities which would prevent proper placement and compaction of core material will be scaled or treated with dental concrete. As the embankment comes up in elevation

against the abutments, rock surfaces under the core and filter zones will be recleaned and detailed surface treatment applied immediately prior to placement of embankment materials.

Spillway

GENERAL EXCAVATION

12-24 Based upon exploratory data collected thus far, it is estimated that at least 90 percent of the spillway can be excavated by mechanical means. The results of the proposed test excavation from Government Canyon Ridge are needed to verify and quantify this assumption. Spillway cut slopes on all but the hydraulic control section at the bottom of the spillway excavation can also be shaped by mechanical means. Isolated blasting of hard rock areas may be required.

SLOPE AND INVERT CONTROLLED EXCAVATION

12-25 Controlled blasting will probably be required on the lowermost spillway slopes in the hydraulic section of the spillway and for the excavation of the spillway invert. If blasting is required, test blasts in non-critical areas during construction will determine the proper method of controlled blasting. Test blasts in Government Canyon Ridge during the test excavation will provide guidance for preliminary planning.

DISPOSITION OF MATERIALS

12-26 Subject to verification during the test excavation and test fill contract, it is expected that, after stripping of topsoil and organic matter, all materials excavated from the spillway would be used in the construction of the embankment without processing. Oversize rocks, if encountered, would be removed individually during loading of the materials. Any zones of particularly soft or highly fractured material would be evaluated for suitability during construction and directed to be wasted or placed in miscellaneous fill areas if unsuitable for use in the embankment.

SLOPE REINFORCEMENT

12-27 Because of the expected variable orientation of the discontinuities in the spillway excavation it is likely that portions of the spillway slopes will require overexcavation, slope reinforcement or both. The method of slope stabilization, whether it be rock bolts, wire mesh, shotcrete or reconfiguration, will be determined during construction.

SILL CONSTRUCTION

12-28 Rock conditions at the invert sill will very likely be identical to those encountered during construction of the spillway invert and slopes in the hydraulic control section. Controlled excavation will be

required to reduce to the maximum extent possible the disruption of the in situ rockmass. Vertical anchors will be drilled and grouted in place in the bottom of the sill trench excavation and tied to the steel reinforcement within the sill.

Drainage Channel

12-29 Excavation requirements for the drainage channel through Government Canyon Ridge are the same as for the spillway, except that no control sill would be required. Materials excavated from the drainage channel, including overexcavation in the upstream direction, will likely be used in the lower elevations of the embankment, with spillway excavation materials reserved for the higher elevations in order to minimize transportation costs.

Embankment

12-30 Criteria for placement of embankment materials are preliminary and are subject to change as further information becomes available during the FDM study phase. Core material requirements are based upon laboratory testing and will be confirmed in a verification fill to be conducted by the contractor during the initial phase of the embankment construction contract. Alluvial filter and shell materials and rock materials from the spillway and drainage channel excavation will be evaluated in a test excavation and test fill contract to be conducted during the FDM study period. See plate A-99 for the estimated quantities and distribution of materials from each source.

IMPERVIOUS CORE

12-31 Materials from the impervious borrow area would be used for construction of the central core of the embankment, and for the slurry trench cutoff backfill and upstream impervious blanket proposed for diversion and control of groundwater during excavation of the foundation.

Material Properties

12-29 To meet design requirements for the embankment, impervious materials should have low permeability, minimal compressibility, be resistant to cracking, and be relatively deformable under large strains. To meet these goals, the borrow materials will be required to have at least a specified minimum percent of fines for low permeability, and will be compacted to a relatively high percentage of maximum density to minimize post-construction settlement and cracking due to differential settlement. To retain acceptable deformability, the materials would be compacted slightly wet of optimum moisture content. Impervious materials placed in contact with foundation bedrock would receive additional moisture to increase deformability and improve bonding with the bedrock. Slurry trench backfill materials would be

mixed with bentonite slurry before placement in the slurry trench. The materials used for the impervious blanket used to connect the slurry trench with the cofferdam would be treated the same as the core materials in the embankment.

Excavation, Transportation and Processing

12-32 Excavation of the impervious core borrow area can be accomplished with conventional heavy construction equipment. After clearing and grubbing to remove irrigation structures, topsoil and vegetation, including citrus orchards, the borrow area would be excavated to an average depth of 15 feet. Since, generally, the surficial soils are dry and the amount of fines decreases with depth, prewetting and vertical excavation cuts would be required to blend the various soil layers and produce a nearly uniform core material within the specified moisture and gradation ranges.

Lift Thickness and Compaction

12-33 A procedural specification for compaction of impervious materials will be used to obtain the design density and moisture content, with the method evaluated in a verification fill to be conducted by the Contractor at the start of the embankment construction. Based on laboratory tests and experience with similar materials, a maximum lift thickness of 12 inches will be specified along with 8 passes of a 50-ton rubber-tired roller, or 8-inch lifts and 8 passes of a tamping roller.

FILTER

12-34 Minus 3-inch alluvial materials for the filter zones will be produced by passing excavated materials from the foundation and the downstream pervious borrow area over a grizzley to remove oversize stones.

Material Properties

12-35 The downstream filter zone must have a gradation after placement and compaction that meets the filter criteria described in section IX in order to prevent migration of core materials due to through seepage. The upstream filter zone must be comprised of cohesionless materials that cannot sustain an open crack so that they will collapse and wash into any crack that occurs in the adjacent core zone, thus plugging the crack and reducing the velocity of flow through the cracked section.

Excavation, Transportation and Processing

12-36 Excavation procedures in the downstream pervious borrow area to produce filter and alluvial transition materials are dictated by the coarse nature of the alluvial materials, the range of particle sizes which can be economically transported and be satisfactorily placed in the embankment, and the need to maintain the capability of the area to function as a major groundwater recharge operation. Because the most economical transportation system appears to be belt transportation, with

a maximum particle size capacity of approximately 12 inches, the excavation plan is geared toward producing a product that can be belt conveyed and is also suitable as embankment fill. Toward this end, it is assumed for estimating costs that, after clearing and grubbing of vegetation, spreading basin gate and drainage structures, and abandoned building foundations, the materials would be pushed with dozers to portable grizzlies which would separate the plus and minus 12-inch materials. A portion of the minus 12-inch material would be passed through an additional grizzley screen to produce a minus 3-inch material that would be used in the upstream and downstream filter zones. The minus 3-inch and minus 12-inch materials would alternately feed directly onto a belt transportation system leading to the damsite, and the plus 12-inch rock would be stockpiled and reloaded into trucks for transportation to the dam. The dozers would push materials from within a maximum radius of about 150 feet to the grizzley, after which the separation plant would be moved to a new location. It is estimated that four of the portable grizzlies would be required to meet production quantities, with two operating and two being moved and set up at any one time.

12-37 For efficient operations, the depth of excavation should be at least 30 feet, and the conceptual borrow area plan is based on this requirement, as well as attempting to avoid unnecessary impacts on the endangered plants in the wash, and the need to maintain drainage and the functional operation of the overall area as a commercial water spreading operation. A borrow plan utilizing an average excavation depth of 40 feet and a typical cross section of the proposed excavation scheme are shown on plate A-55. They show that, to provide flood protection to the borrow area and enable efficient excavation depths to be utilized as much as possible, a berm would be constructed next to the Santa Ana River streambed at the southern border of the borrow area so that the excavation could be carried below the elevation of the adjacent stream thalweg. The berm, if necessary, would be less than 15 feet high and could be constructed with oversize or waste material from borrow and embankment construction operations. In order to maintain drainage and the operation of the percolation basins during the estimated 4 years of borrow operations, borrow excavation is proposed to be phased in a sequencial order as shown on plate A-101. After a significant area has been excavated, replacement dikes and gate structures would be constructed so that water spreading operations could be shifted from an area that is next to be excavated. Since no more than about one-fourth to one-third of the borrow area would be excavated in any one year, a minimum of one-half of the area would be continuously available for continued water spreading operations.

12-38 Groundwater levels in the proposed borrow area have been monitored by the San Bernardino Valley Water Conservation District with 3 observation wells, shown on plate A-55 as wells 7B1, 7K1 and 12J1. These well records indicate that the depth to groundwater within the borrow area limits has varied in the last 10 years from 10 to 250 feet in the southern and western portions and from 48 to 300 feet near the northern boundary. Currently, the depth to water is between 230 and

280 feet in these wells after a year of very low rainfall. If the area experiences 1 or 2 years of relatively high rainfall, the groundwater levels could rise to within the proposed borrow depths, causing delays in excavation and increases in construction costs.

Lift Thickness and Compaction

12-39 Based on experience with similar materials on numerous projects within the Los Angeles District, a maximum lift thickness of 12 inches will be specified to minimize the segregation tendencies of the filter material. Compaction would be accomplished with 4 passes of a vibratory roller, with water added to facilitate compaction.

ALLUVIAL TRANSITION

12-40 Alluvial transition materials for the upstream portion of the embankment will be produced from excavation of the foundation and the downstream pervious borrow area. An additional borrow source, if required, would be the streambed alluvium upstream of the dam to the confluence with Warm Springs Canyon.

Material Properties

12-41 The alluvial transition materials must have high strength, resistance to erosion and deformation, and be relatively free draining. Adequate strength, erosion resistance and deformation resistance can be obtained by compacting the naturally hard alluvial particles to high degrees of compaction. Compaction above the critical void ratio, combined with the relatively high permeability if the amount of fines is limited, will cause the materials to dilate and prevent high induced pore pressures and resultant loss of strength during earthquake shaking.

Excavation, Transportation and Processing

12-42 Excavation, transportation and processing of alluvial transition materials from the downstream pervious borrow area have been described in the preceding paragraphs on filter materials. Materials obtained from the foundation excavation and upstream pervious borrow area would be excavated with a combination of dozers and loaders and hauled to a grizzley plant in offroad trucks. The materials would be screened on the grizzley to produce plus and minus 12-inch materials, and hauled back to the embankment in trucks.

Lift Thickness and Compaction

12-43 The lift thickness and compaction requirements for the alluvial transition material will be determined by evaluation of the test fill to be conducted during the FDM study phase. Lift thicknesses of 18, 24 and 30 inches will be tested with 12 and 18 inch maximum stone sizes. Vibratory steel-wheeled and rubber-tired rollers will be evaluated with varying numbers of passes for each lift thickness.

DOWNSTREAM TRANSITION

12-44 Materials for the downstream transition zone will be produced by excavation of the spillway and the drainage channel through Government Canyon Ridge. Materials excavated from the outlet works tunnel and the abutments will also be evaluated for suitability as embankment fill materials for this zone. Since the drainage channel represents a source of low elevation materials, compared to the spillway, it is expected that the initial downstream transition construction would utilize these materials. After the embankment elevation is increased, it will become more cost effective to begin hauling spillway materials to the embankment.

Material Properties

12-45 The properties of the materials to be used in the downstream transition will be investigated in the test fill contract during the FDM study phase. Preliminary evaluation based on drill logs and rock quality tests on drill cores indicate that the construction materials are likely to be relatively compressible and impervious compared to the alluvial transition materials in zone 3. Variation in particle size and rock quality during excavation is expected to result in a corresponding variation in strength and compressibility in the fill materials.

Excavation, Transportation and Processing

12-46 Excavation of the spillway and drainage channel is discussed in detail in paragraphs 12-24 through 12-29. It is anticipated at this stage of investigation that processing of the excavated materials will not be required to produce acceptable embankment fill. Occasional oversize stones or seams or zones of soft rock will be evaluated when encountered and wasted if not suitable for miscellaneous or embankment fill. For estimating purposes, it is assumed that the excavated materials would be loaded into offroad trucks with front end loaders and hauled to the dam.

Lift Thickness and Compaction

12-47 The lift thickness and compaction requirements for the downstream transition materials will be determined by evaluation of the test excavation of Government Canyon Ridge and test fill to be conducted during the FDM study phase. The excavation and test fill program will investigate the compaction characteristics of materials produced by varying excavation and compaction methods. For estimating purposes, it may be assumed that the material would be spread in 12-inch thick lifts and compacted with 4 passes of a rubber-tired or vibratory steel-wheeled roller.

SHELL

12-48 Materials for the outer shells of the embankment would be produced by processing alluvial materials through a grizzley to remove the minus 12-inch material and removing plus 30-inch stones individually.

Material Properties

12-49 Oversize stones from the pervious borrow, upstream borrow and foundation excavation would be limited to a maximum diameter of 30 inches in order to facilitate handling.

Excavation, Transportation and Processing

12-50 The oversize shell materials will be excavated and processed as described in the section on filter materials. Stone in the stockpiles in the borrow areas will be loaded into large offroad trucks, hauled to the embankment, and spread by bulldozers. After spreading, a layer of smaller alluvial or abutment excavation material, with a maximum particle size of 12 inches would be placed over the shell materials to improve trafficability. The traffic layer would not be permitted in the outer few feet of the zone since that portion is also providing slope protection.

Lift Thickness and Compaction

12-51 The estimated lift thickness for shell materials with diameters ranging from 12 to 30 inches is 36 inches. Compaction would be accomplished with vibratory rollers and supplemented by the action of the bulldozers during spreading and the controlled movement of haul trucks over the fill.

Roads

CONSTRUCTION ROADS

12-52 Design and construction of temporary construction roads in accordance with applicable safety standards will be the responsibility of the Contractor. The Corps will delineate haul route corridors and the location of utilities that need to be protected. Erosion control measures acceptable to the U.S. Forest Service will be required on all roads, as will mitigation for environmental damage.

Truck Transportation Haul Roads

12-53 Haul roads for truck transportation of embankment fill materials and for equipment and personnel access will be required to bring material from the borrow sources and deliver it to various elevations on the embankment. Traffic control will be required where the haul road from the downstream pervious borrow area crosses Greenspot Road. A common road up the streambed from the canyon mouth can be used for materials from the impervious and downstream pervious borrow areas. Temporary roads that are constructed along the canyon walls will have maximum cut slopes of 4V on 1H in rock and 1V on 1H in soil. Maximum fill slopes would be 1V on 1.5H. Haul roads and temporary access roads will be constructed with materials from required excavation for the roads or with miscellaneous fill materials that are not suitable for embankment fill.

Belt Transportation Maintenance Roads

12-54 If minus 12-inch materials from the downstream pervious borrow area are transported to the damsite on a belt system, a maintenance road next to the belt will be required. The road would be approximately 15 feet wide and would be constructed with required excavation for the road or miscellaneous fill materials not suitable for embankment fill.

PERMANENT ACCESS ROADS

12-55 Permanent access to the top of the embankment and the reservoir area will be provided by an all-weather asphalt-paved road from Alder Creek Road to the top of the dam at the right abutment, across the top of the dam, and then past the spillway to Warm Springs Canyon, where it will connect with an existing access road to the stream bottom area. The road would be designed in accordance with the requirements in TM 5-822-5, "Engineering and Design, Flexible Pavements for Roads, Streets, Walks, and Open Storage Areas."

Materials

12-56 Suitable materials for the road base would be available from required excavation for the road. Asphalt materials for the surface layers would be obtained from commercial sources in the vicinity.

Placement Requirements

12-57 Subgrade, base course and surface materials would be placed and compacted in accordance with TM 5-822-5. Permanent cut slopes would be 4V on 1H in rock and 1V on 2H in soil. The maximum fill slope would be 1V on 1.5H.

Relocations

ROADS

12-58 Alternate access to the Southern California Edison facilities in Santa Ana River Canyon must be provided when construction of Seven Oaks Dam begins. Design and construction of the roads will be a project requirement. Investigations and detailed design of the roads will be accomplished during the FDM study phase.

POWERLINES

12-59 Relocation of Southern California Edison powerlines in the reservoir area will be required.

XIII. CONSTRUCTION SCHEDULE

13-01 The construction schedule for Seven Oaks Dam is influenced by significant design investigations for the main embankment, spillway, and outlet works that remain to be completed during Feature Design Memorandum studies. The Phase II General Design Memorandum investigations, however, have enough information to be able to prepare plans and specifications for two preliminary construction contracts that would significantly shorten the time required to construct the dam and appurtenant structures once the FDM's are completed. The two preliminary construction contracts include the test excavation and test fills contract, and the abutment excavation contract. These preliminary construction contracts, as well as the outlet works FDM and construction, will need to be completed in a timely manner during the FDM study phase to provide data for the design and analysis, to provide information for the main construction contract, and to reduce overall costs of construction by reducing the time required for construction. The construction schedule and work associated with these contracts, in addition to the outlet works and the embankment and spillway construction schedules, are discussed in this chapter.

Test Excavation and Test Fills

GENERAL

13-02 A detailed investigation of alluvial materials in the dam foundation, a test excavation of bedrock along Government Canyon Ridge, and test fills with alluvial foundation and bedrock materials are the main components of the first construction start. Impervious core material would not be included in the test fill contract since it would involve constructing haul roads from the borrow area and developing the borrow area well in advance of construction of the dam. Instead, a verification fill using core borrow material is proposed to take place during the beginning stages of the main embankment contract. Furthermore, the core materials are easily tested in the laboratory. In contrast, the test excavation and test fills contract represents a multi-purpose investigation of materials which cannot readily be tested in the laboratory. The intent of this contract is to gain both

quantitative and qualitative information on foundation conditions, construction methods, material properties, quantities, cost, and quality control testing methods. The detailed information obtained from the test excavation and test fills is required to complete the analysis and design, and for economical construction of the dam. Results of these investigations would be presented in the Embankment and Spillway FDM due to be completed in 1992. The features of the investigation program incorporated in the test excavation and test fills contract are discussed in the following paragraphs.

DAM FOUNDATION INVESTIGATIONS

13-03 The purpose of the investigation of alluvial foundation materials would be threefold: (1) to obtain material properties required to further evaluate the foundation conditions and to be used in the static and dynamic analyses for the FDM, (2) to evaluate construction methods in order to incorporate such information in the preparation of the embankment contract, and (3) to obtain a sufficient quantity of materials to construct the alluvial test fill. In order to accomplish these goals a variety of field and laboratory tests would be conducted to obtain both quantitative and qualitative information on the foundation conditions. The field testing, as subsequently described, would consist of geophysical testing, foundation excavation, and in situ testing. Laboratory testing would consist of particle size and large-scale vibrated maximum density determinations.

13-04 Geophysical testing would be performed at the locations of three existing sets of crosshole borings in the streambed within the limits of the embankment. These borings have been drilled, cased and grouted in the embankment foundation to the specifications of ASTM D 4428 for crosshole seismic testing. The crosshole sets are located near the upstream and downstream toes and near the centerline of the dam as shown on plate A-102. All holes were drilled to bedrock at depths of 90 feet or greater with a rotary wash drill rig. A profile of shear wave velocity will be developed from the test results. An evaluation of the test results to determine dynamic moduli and relative densities within the foundation will be made.

13-05 At the location of the three crosshole sets, the foundation would be investigated by excavating large test pits to a depth of 40 feet. The test pits would have 20 by 20 foot base dimensions, 1V on 1H side slopes on three sides, and 1V on 6H access ramps, and would each yield approximately 15,600 yd^3 of excavated materials. Large-scale in situ density testing and gradation sampling would be conducted at 5-foot depth intervals within the excavation thus providing information on the foundation conditions in the critical areas near each toe. Comparison of in situ densities and corrected laboratory large-scale vibrated maximum densities would assist in the evaluation of foundation conditions. A profile of gradation and density varying with depth will augment the geophysical test results and assist in characterizing the foundation conditions. In addition, the development of relationships between the properties of the matrix fraction and those of the total in situ sample will be investigated as a possible means of evaluating the foundation conditions.

13-06 Because the groundwater level is normally about 10 to 15 feet below the surface, the test pits would need to be dewatered by drilling approximately 5 wells around each site and installing submersible pumps. To obtain information on both saturated and drained conditions, crosshole geophysical surveys would be conducted before and after drawdown of the groundwater surface. By measuring drawdown rates in adjacent wells, additional permeability values for the foundation would be obtained. Materials excavated for the density tests would be used to determine the matrix gradation since boulders larger than 1 foot in diameter are difficult to excavate by hand. Visual observations of the stratification within the test pit would augment the matrix gradations in order to better describe the foundation materials.

ALLUVIAL TEST FILL

13-07 The purpose of an alluvial test fill would be (1) to evaluate construction methods which could be used in the construction of zone 3 of the embankment; (2) to observe the behavior of alluvial materials during placement and compaction; (3) to form a basis for determination of design values for the alluvial materials used in construction; and (4) to provide more detailed information on material quantities and cost. In addition, information would be obtained to evaluate whether foundation conditions could be improved by excavating the in situ alluvium and replacing it with compacted fill.

13-08 The alluvial test fill would be constructed using representative materials excavated from the test pit and test fill foundations. Oversize rocks would be removed from the excavated materials by passing the materials through a grizzley prior to being placed in the test fill. The remaining minus 12-inch and minus 18-inch materials, approximately 30,000 yd^3, would be placed in the test fill in varying lift thicknesses and then compacted with varying levels of compactive effort as illustrated on plate A-102. The compaction equipment specified to be supplied for the alluvial test fill would include 5- to 10-ton and 15- to 20-ton steel wheel vibratory rollers. Large-scale in situ density tests would be conducted in the test fill to evaluate compaction and segregation associated with various maximum particle sizes and lift thicknesses. Survey control points on the test fill would be measured periodically to evaluate the effects of compaction on the various fill materials. Trenches would be excavated through the various layers upon completing the test fill to evaluate the condition of the compacted layers and to further check for segregation of particles. Densities obtained in the test fill would be compared to those of the in situ foundation and rock-corrected laboratory maximum densities. Additional comparisons would be available by drilling a set of crosshole borings in the completed test fill and conducting geophysical surveys. If practical, field permeability tests would be conducted in the test fill to obtain parameters to be used in the detailed stability analysis of the embankment. A schedule of proposed tests associated with the alluvial test fill is shown in table A-68. Results obtained from the test fill would be incorporated in the preparation of construction specifications for the embankment zones to be constructed with alluvial materials.

TEST EXCAVATION AND ROCK TEST FILL

13-09 In conjunction with the alluvial test fill, a test excavation and test fill would be conducted to evaluate the bedrock materials that would be produced by the required excavation of the spillway and drainage channel. Geologic investigations and laboratory tests indicate that the bedrock is relatively soft and highly fractured. The investigations suggest that these materials may degrade during excavation and compaction to a relatively poor draining, compressible embankment fill that does not meet the classification requirements for a rockfill. A test excavation in the area of the drainage channel through Government Canyon Ridge is proposed to obtain information on cost and feasibility of different excavation methods, and to obtain materials for the test fill. The purpose of the rock test fill is to evaluate the behavior of the rock fill and construction methods. The compaction equipment specified to be supplied for the rock test fill would include a 5- to 10-ton and a 15- to 20-ton steel wheel vibratory roller and a 50-ton pneumatic tire roller. A plan and typical section of the test exca-vation and rock test fill is shown on plate A-102. The drainage channel location was selected over the spillway site due to accessibility considerations which significantly influence the cost of the test excavation.

13-10 The drainage channel excavation would consist of removing the more weathered rock near the surface, down to elevation 2,325, and using it to begin construction of a haul road down Government Canyon to the test fill site in the streambed. With the floor of the excavation at elevation 2,325, two or three exploratory wireline core holes would be drilled within the test excavation area. The quantitative rock quality data from the drilling would be used in evaluation of the subsequent test excavation and test fill data. Materials removed between elevation 2,325 and 2,260 would be excavated partially by drilling and blasting and partially by mechanical ripping to provide approximately 15,000 yd^3 for the rock test fill. On the basis of exploration conducted thus far, the portion below elevation 2,300 is thought to be representative of the majority of rock that would be excavated from the spillway and Government Canyon Ridge. The material above elevation 2,300 is believed to be somewhat inferior in quality, however, a portion of this rock would also be evaluated in a test fill. The test fill would evaluate the parameters of lift thicknesses, compactive effort, segregation, permeability, and particle breakdown. A schedule of proposed testing of the rock test fill is shown in table A-69. The results would provide a basis for determination of design values, embankment zonation and construction specifications.

13-11 In order to evaluate the settlement characteristics of the foundation and test fill materials, the top 10 feet of streambed alluvium would be removed in the area of the test fills and the foundations prepared in a manner similar to what is anticipated for the remainder of the embankment. These excavated materials would be used with the foundation test pit materials in the alluvial test fill, which would be constructed adjacent to the rock test fill. Lessons learned in

the excavation and preparation of the test fill sites would be applied to specifications for construction of the dam. If the completed rock test fill does not meet the requirements for embankment fill, it would be removed as part of the main dam construction contract.

OUTLET WORKS PORTAL EXCAVATION

13-12 To enable exploratory adits to be excavated at the intake and outlet portals for the outlet works, equipment mobilized for test fills would also be used to excavate the colluvial and alluvial overburden at these locations. A total of approximately 40,000 yd^3 would need to be removed to expose the bedrock at the elevation of the proposed portals. The approximate limits of the excavations are shown on plate A-102. Because of expected impacts, relocation of the graves near the outlet portal and the Southern California Edison access road at both portals will be necessary before excavation can be initiated.

OUTLET WORKS ADITS

13-13 An adit will be excavated at each portal during the FDM stage. The upstream adit will be 500 feet long and the downstream adit 200 feet long in rock. Both adits will be a nominal 6 feet wide by 7 feet high supported where necessary. Also, both will be located within the proposed outlet works excavation so the adits will not interfere with any of the final structures. The purpose of the adits is to determine the joint and other fracture patterns and the presence of significant shears for excavation and support design in the critical portal areas. Because of the high regional tectonic activity, abnormal residual stresses could be present which could affect the tunnel support design. At least one of the adits will be used to determine possible stresses by means of flat jack testing. The adits will also provide a means for bidders for the outlet works construction to visually inspect site conditions.

STREAMBOTTOM EQUIPMENT ACCESS ROAD

13-14 An access road suitable for mobilization of equipment for the test excavation and test fills would be constructed as the first part of this contract. Beginning at Greenspot Road, just north of the Santa Ana River bridge crossing, a new road will be constructed along the streambed to connect with the existing exploration access road that starts at the paved road next to SCE powerhouse No. 3 at the mouth of the canyon. A concrete encasement will be constructed to protect the flume carrying water from the powerhouse to the spreading basins. The current exploration access road between the mouth of the canyon and the damsite will be regraded and widened to allow safe use by the higher volumes of traffic expected during the test fill construction.

13-15 Construction of the test fills contract work would require 4 to 6 months to complete. According to the present schedule, about 12 months would remain in which to complete laboratory testing, evaluate the data, incorporate results into the design and analysis, and present

them in the Feature Design Memorandum for the embankment. The plans and specifications for construction of the dam would be prepared after approval of the FDM.

Outlet Works

13-16 Construction will begin with additional subsurface investigations and development of the final design. Following completion of the Feature Design Memorandum and development of the plans and specifications actual construction will commence. The construction will be divided into two phases; the first phase sufficient for flow diversion during construction of the embankment and the second phase with installation of the regulating and other mechanical and electrical items. The initial phase, which includes all excavation, completion of the channels and energy dissipator, lining of the tunnel and construction of the basal portion of the inlet and outlet structure, will be completed in the spring of the following year. Following a hiatus until the construction of the embankment begins, phase two will be completed. Bar charts depicting the outlet works construction schedule are presented in the Main Report of Volume 1.

Abutment Excavation

13-17 A construction contract to excavate the majority of the overburden soils and loose rock on the abutments in advance of the main embankment construction is scheduled to follow the test fill activities. It is designed to reduce the time required to complete the embankment construction after completion of the FDM, and to provide additional information for preparation of the plans and specifications for the dam. Approximately 6 months would be required to complete the abutment excavation. A detailed description of the construction requirements for this contract is in section XII of this appendix.

Embankment and Spillway

13-18 The main construction contract for the embankment and spillway is scheduled to begin after completion of the embankment FDM. The estimated schedule of the various stages of construction discussed in the following paragraphs assumes that the abutments have been excavated in a previous contract and that the outlet tunnel is capable of being used for diversion and control of surface flows.

DIVERSION AND CONTROL OF WATER

13-19 Diversion and control facilities, including the slurry trench cutoff and the cofferdam, would be constructed during the winter of the first year of the construction contract so that below streambed construction could begin at the start of the next dry season. During this period the Contractor would mobilize equipment, construct access

and haul roads, and prepare processing equipment for borrow operations. Access to the area upstream of the dam must be provided before excavation of the foundation trench can commence.

FOUNDATION EXCAVATION

13-20 Foundation excavation can begin at the start of the dry season if protection from flood flows is provided by the outlet works tunnel and cofferdam. Completion of the cofferdam to design height is required as soon as possible to reduce the risk of inundation of the construction site. Excavation of streambed materials down to bedrock within the central area and to a depth of 20 feet over the remainder of the embankment footprint is expected to take approximately three months.

FOUNDATION TREATMENT

13-21 Treatment of the exposed abutments and foundation will begin as soon as possible after being exposed by the streambed excavation in order to complete the below streambed activities, including surface treatment, drilling and grouting, and embankment fill in the shortest practical time, thereby reducing the risk of damages and delays from flood flows. Foundation treatment in the bottom of the trench is expected to be completed in approximately 4 months, so that embankment fill materials can begin to be placed.

BELOW STREAMBED EMBANKMENT FILL

13-22 Below streambed embankment fill will begin as soon as curtain grouting and surface treatment of the bedrock surface is completed in the bottom of the foundation trench. Because of the restricted work space and special requirements for initial lifts of impervious materials, production rates will be relatively low during the early stages of below streambed embankment fill operations. It is estimated that a reasonable time required to regain streambed elevation after commencement of foundation excavation would be approximately 8 to 12 months.

ABOVE STREAMBED EMBANKMENT FILL

13-23 The construction schedule for the remainder of the dam is based on the assumption that fill above the streambed level can be placed at an average rate of 1,000,000 yd^3 per month. Estimates indicate that this rate could be accomplished by working 2 10-hour shifts per day, 6 days per week.

Table A-68. ALLUVIAL TEST FILL, TESTING SCHEDULE

Lane Number	Lift Number	Lift Thick. (in.)	Settlement Measurements	Mechanical Analysis Before Rolling	Mechanical Analysis After Rolling	In Situ Density Tests	Vibrated Lab Density Tests	Other Field Tests
Foundation	-	-	X	3	6	6	3	A
1,2	1A	18"	X	4	4	16	4	B
	1B	18"	X	4	4	0	2	B
	1C	18"	X	4	4	0	2	B
	1D	18"	X	4	4	0	2	A,B
1,2	2A	24"	X	4	4	16	4	B
	2B	24"	X	4	4	0	2	B
	2C	24"	X	4	4	0	2	B
	2D	24"	X	4	4	0	2	A,B
1,2	3A	30"	X	4	4	16	4	B
	3B	30"	X	4	4	0	2	B
	3C	30"	X	4	4	0	2	B
	3D	30"	X	4	4	0	2	A,B
1,2	4	36-48"	X	2	4	4	2	A,C

LEGEND

A = IN SITU PERMEABILITY TEST
B = PLATE LOAD TEST
C = GEOPHYSICAL TESTING

Table A-69. ROCK TEST FILL, TESTING SCHEDULE

Lane Number	Lift Number	Lift Thick. (in.)	Settlement Measurements	Mechanical Analysis Before Rolling	Mechanical Analysis After Rolling	In Situ Density Tests	Vibrated Lab Density Tests	Other Field Tests
Foundation	-	-	X	3	3	6	3	A
1,2,3	1A	9"	X	6	6	18	3	B
	1B	9"	X	6	6	0	3	A,B
1,2,3	2A	12"	X	6	6	18	3	B
	2B	12"	X	6	6	0	3	B
	2C	12"	X	6	6	0	3	B
	2D	12"	X	6	6	0	3	A,B
1,2,3	3A	18"	X	6	6	18	3	B
	3B	18"	X	6	6	0	3	B
	3C	18"	X	6	6	0	3	B
	3D	18"	X	6	6	0	3	A,B
1,2,3	4A	24"	X	6	6	18	3	B
	4B	24"	X	6	6	0	3	A,B,C

LEGEND

A = IN SITU PERMEABILITY TEST
B = PLATE LOAD TEST
C = GEOPHYSICAL TESTING

XIV. QUALITY CONTROL

Quality Control Testing Program

14-01 The quality control testing program for Seven Oaks Dam would consist of material acceptance tests, construction quality control tests, and record tests. Acceptance tests are conducted on materials used in appurtenant structures and deal with materials such as metals, waterstops, joint sealants, and some materials associated with concrete construction. Materials acceptance tests exceed the capability of local laboratories and are conducted by either the South Pacific Division laboratory or the Waterways Experiment Station and the test results determine the acceptability of the material for use in the construction. Upon developing feature design details, the scope of acceptance testing would be determined. The following paragraphs describe construction quality control tests and record tests as they relate to embankment construction.

Construction Quality Control Testing

CONTRACTOR QUALITY CONTROL

14-02 The Contractor will be required to provide quality control testing of materials that are placed in the embankment in order to show that they meet specification requirements. Testing will be conducted before compaction. A description of tests for each material is provided in this section.

GOVERNMENT QUALITY ASSURANCE

14-03 The Government will provide inspection of construction operations and quality assurance testing of embankment materials after compaction to insure that design assumptions for foundation conditions, density, seepage, strength and consolidation are adequately met. The quality assurance testing can be accomplished with Government personnel or a combination of Government and private industry personnel as described below.

Government Staff

14-04 Inspection of geotechnical construction operations critical to the safety and performance of the embankment will be conducted by experienced personnel familiar with the design criteria for the embankment. These operations include outlet works excavation, slurry trench backfill, foundation grouting, surface treatment, placement and compaction procedures, and installation of instrumentation equipment. To assist in the inspection and evaluation of construction operations, the Los Angeles District Geotechnical Branch will provide a project geologist and an embankment engineer who are familiar with the project design as technical assistants to the Resident Engineer.

14-05 The Government quality control testing staff will include, at the very least, a supervisor of laboratory testing for the embankment materials and an assistant to supervise the expected second shift operations. These personnel will need to be experienced in soil testing and construction procedures, and proficient in compiling and reporting test data in a timely manner so that conditions that may adversely effect the embankment safety are adequately monitored. Additional personnel to perform field and laboratory testing can be provided by Government employees with soil testing experience, either civilian or regular army, or, if not readily available, by contract with a private soils testing firm as described in the following paragraph.

Contract Testing Laboratory

14-06 Trained technicians with soils testing experience can be provided to perform field and laboratory tests by contract with a private soils testing firm as was done at Warm Springs Dam. A description of the tests required and the expected frequency of testing was the basis for an advertised contract where the bidders were required to submit a competitive bid to provide personnel to perform the tests at a unit price per test. This method allowed for a reasonable variation in number and frequency of testing due to construction schedule shifts and weather delays. The testing personnel were supervised by the Government laboratory director to provide tests necessary for quality assurance testing of the embankment. For maximum flexibility, a contract would be for a one year period with a provision included in the contract to extend it for an additional period if results are satisfactory and it is mutually agreeable to both parties.

TESTS

14-07 The following summary of construction control tests is preliminary and subject to change upon further evaluation during the FDM design phase. The test fill, scheduled to be constructed as the first construction start at Seven Oaks Dam, is expected to be the basis for material testing and construction procedure specifications for all but the filter and impervious core materials.

Core

14-08 Before compaction, impervious core materials delivered to the embankment will be tested by the Contractor for compliance with specifications for maximum particle size, gradation limits and moisture content. Moisture-density relationships will be developed through a family of five-point compaction curves to determine maximum density and optimum moisture values for the range of core materials. After compaction, in situ density and moisture content will be determined through Government quality assurance testing, primarily with sand cone density tests. Nuclear methods will be used for quick tests of density and moisture contents only. Relative compaction values will be determined with a one-point or three-point compaction test for every sand cone test, and a smaller number of five-point compaction tests. Gradation and Atterberg limits tests will be used in conjunction with previous laboratory consolidation tests to evaluate the expected settlement of the core.

Filter

14-09 Filter materials, consisting of minus 3-inch alluvial particles, will be tested by the Contractor for compliance with maximum particle size, gradation and uniformity limits before compaction. Evaluation of in-place materials by the Government testing staff will include gradation and density tests, and a critical inspection of filter material distribution after spreading operations to preclude procedures that lead to excessive segregation. In situ density tests will be conducted with either 12-inch diameter sand cones or large-scale, water replacement density testing equipment. Maximum density values for the filter materials will be determined using vibrated test methods with a surcharge, either in an 11-inch diameter mold according to ASTM D 4253 or in the District's 27-inch diameter mold described earlier in this appendix.

Alluvial Transition

14-10 The alluvial transition materials to be placed in the upstream portion of the embankment will consist of minus 12-inch sand, gravel and cobble size particles from the streambed and alluvial outwash below the canyon mouth. Tests by the Contractor before compaction will consist of gradation and maximum particle size determinations to insure compliance with specification limits. After compaction, large-scale water replacement density tests would be performed by the Government staff and compared to vibrated maximum density tests on the same material to determine relative compaction values. Maximum density tests would be performed on minus 6-inch materials from the field density test with the District's 27-inch diameter mold.

Downstream Transition

14-11 Gradation tests would be performed by the Contractor on the materials from the spillway and Government Canyon Ridge excavation before compaction. After compaction, field and laboratory density tests

would be performed by the Government to determine relative compaction, and additional gradation testing would be used to evaluate particle breakdown during handling and compaction.

Shell

14-12 No tests would be performed on the shell materials due to the overall large size of the cobbles and boulders involved. An inspection of lift thickness, general particle size limits, and particle size distribution would be used to evaluate the shell. The traffic layers of minus 12-inch alluvial or downstream transition material will be used only to fill in the void space between the larger boulders, and no testing of this material will be required.

Reporting

14-13 The Contractor's quality control supervisor will be required in the contract specifications to report the results of laboratory and field tests within various time limits after the testing so that corrective action may be taken if required. The Government quality control supervisor will compile the results of both the Contractor's and Government's testing and supply such information to the Resident Engineer so that the compliance with specification limits and design intent can be evaluated by the Resident Engineer and the Embankment Engineer. Summaries of test data will be transmitted monthly to the District and Division offices in accordance with ER 1110-2-1925, "Field Control Data for Earth and Rockfill Dams".

Monitoring

14-14 The expected large volume of field and laboratory test data to be generated during construction of the embankment will require a computer based monitoring system to keep track of the location and results of the tests. Appropriate computer software and hardware will be used to show the locations of the various field tests. A data base of test requirements and results for each material can be developed using one of the commercially available or in-house data base management programs.

Record Testing

14-15 Record testing of as-constructed embankment materials will be conducted by the South Pacific Division Laboratory on samples obtained by Los Angeles District Geotechnical Branch personnel. Undisturbed cubic foot or large diameter tube samples of the impervious core will be tested to verify design assumptions for horizontal and vertical permeability, consolidation and shear strength. Disturbed samples of the filter and transition materials will be tested to verify gradation, permeability and shear strength of those materials.

XV. INSTRUMENTATION

Preconstruction Microseismic Network

15-01 The seismic network to establish pre-impoundment seismicity, as recommended by Dr. Clarence Allen, in attachment A-1, will be expanded in conjunction with the USGS during the FDM study period. Indications of induced seismic events are primarily determined by deviations in the seismic pattern as a result of activities such as the filling of a reservoir. Expansion of the USGS seismic network by installing a six component seismic station, consisting of two horizontal and one vertical instruments to measure microseismic activity, and two horizontal and one vertical instruments to measure macroseismic activity, is essential in determining the preconstruction seismic pattern. This station will provide the necessary information for determining the focal depth and spatial relationships of seismic activity in the vicinity of the site. Installation of the station as quickly as possible is necessary in order to gather sufficient data to determine the seismic background and pattern of the site prior to construction of the dam.

Embankment

15-02 Instrumentation of Seven Oaks Dam will be consistent with the requirements of EM 1110-2-1908, "Instrumentation of Earth and Rockfill Dams," and ER 1110-2-110, "Instrumentation for Safety Evaluations of Civil Works Projects," and will be appropriate for a large embankment dam in an area of high seismic risk. Specific locations for the various types of instruments will be determined by analysis of the results of the settlement calculations and a finite element analysis of stresses and pore pressures expected within the embankment. The FEM analysis is scheduled to be performed during the FDM study phase. A coordinated program combining construction control sampling and instrumentation observation and recording will be developed to evaluate embankment and foundation performance.

SETTLEMENT

15-03 Vertical movements within the embankment will be measured with a combination of settlement monuments on the surface of the dam and inclinometers installed within the embankment. These monuments will be monitored from offsite survey monuments near the dam that are tied into a regional vertical and horizontal control network. The instrument layout will be designed to monitor the consolidation of the various zones of materials in order to evaluate the effects of differential settlement within the embankment.

HORIZONTAL MOVEMENT

15-04 Horizontal movements on the surface of the dam will be monitored with surface monuments. Horizontal movements within the embankment will be measured with inclinometers and full profile horizontal displacement gauges. The movements will be compared with expected movements predicted by the finite element analysis.

PORE PRESSURES

15-05 Pneumatic type diaphragm piezometers will be installed in the embankment and abutments to monitor pore water pressure during and after construction of the embankment in order to check the validity of design assumptions regarding pore water pressures and to detect potentially dangerous conditions that may affect the stability of the dam. Pneumatic piezometers will be specified because of the very small time lag required for pressure equalization in fine-grained soil, the capability to utilize central observation stations, and because they are less expensive than electrical type diaphragm piezometers. A smaller number of open-tube Casagrande type piezometers will be installed to monitor water pressures after impoundment of the debris pool and to confirm pore pressure readings from the pneumatic piezometers. High air entry filters will be utilized to reduce the tendency to read high pore air pressures instead of pore water pressures. In accordance with the guidance in EM 1110-2-1908, piezometers will be provided at vertical increments not to exceed 80 feet at a minimum of two representative cross sections. Representative soil samples will be collected at the piezometer tip elevations to determine soil characteristics relative to the reading and evaluation of the piezometer data.

SEEPAGE

15-06 In conjunction with piezometer data, readings from observation wells in the abutments and near the toes of the dam will be used to monitor seepage through the embankment and abutments. Measured seepage quantities will be compared with design assumptions of the various conditions of through seepage to evaluate the performance of the dam.

SEISMIC MOTIONS

15-07 An interconnected system of strong motion accelerographs with 1.0-g accelerometers and peak recording accelerographs would be installed in the embankment, alluvial foundation and bedrock abutments to record the response of the dam and foundation to seismic activity. The recorded response at the locations on the abutments, crest, downstream slope and downstream foundation, as suggested in EM 1110-2-1908, will assist in assessment of performance after a significant seismic event. The system will be tied into the network maintained by the U.S. Geological Survey and will conform with the requirements of ER 1110-2-103, "Strong-Motion Instruments for Recording Earthquake Motions on Dams," and ETL 1110-2-195, "Instrumentation for Measurement of Earthquake Motion and Instrument Shelters for Earth and Rock-Fill Dams."

Monitoring Program

EVALUATION DURING CONSTRUCTION

15-08 In order to be able to evaluate the information obtained from the various instruments during construction, bench marks and a reference base line will be established as soon as possible after initiation of the construction contract. All movement measuring devices would be referenced to the established survey points and read immediately after installation and approximately every two weeks thereafter. In order to correlate settlement and piezometric data, piezometers located near movement measuring devices would be read at the same time. Where possible, data will be reduced in the field so that erratic readings can be checked immediately. To clearly establish trends and relationships, instrumentation data will be presented in graphical form.

VERIFICATION OF DESIGN ASSUMPTIONS

15-09 During the construction period, instrumentation data on pore pressures and earth movements within the embankment will be compared with predicted values to verify the assumptions that were made during the design of the dam. Record samples of embankment fill materials will be obtained and tested to determine the in-place characteristics of the various soils in the dam. Significant variation between the two data sets will be analyzed to evaluate the potential effect on the stability of the dam and appurtenant structures.

POST-CONSTRUCTION F.E.M. EVALUATION

15-10 A finite element method analysis of the embankment as built will be performed after construction using material characteristics from quality control tests, and pore pressure, seepage and earth movement data from instrumentation records. The analysis will be used to evaluate the conformance with design assumptions and to predict future performance of the dam. The results will be presented in the "Embankment Criteria and Performance Report."

POST-CONSTRUCTION PERFORMANCE MONITORING

15-11 The monitoring and reporting of instrumentation data from Seven Oaks Dam will continue after construction is completed to enable Los Angeles District personnel to evaluate the continuing performance of the dam. Of particular importance, instrumentation records will form a base line of data to be compared with readings taken during impoundment of flood waters and immediately after a significant earthquake. In conjunction with recorded data, inspections of the embankment and reservoir area will be conducted in these situations to check for unusual conditions such as loss of freeboard by settlement, evidence of through or underseepage, abnormal embankment cracking or slumping, and landslides on the abutments.

XVI. REFERENCES

1. Allen, C.R. (1957) "San Andreas fault zone in San Gorgonio Pass, Southern California," Geologic Society of America Bulletin, v. 68, pp. 315-350.

2. Banerjee, N.G., Seed, H.B., and Chan, C.K. (1979) "Cyclic Behavior of Dense Coarse-Grained Material in Relation to the Seismic Stability of Dams," Report No. UCB/EERC-79/13, Earthquake Engineering Research Center, University of California, Berkeley.

3. Bolt, B.A. (1987) "Seismological Report for Seven Oaks Dam for Santa Ana River Basin, California for Geotechnical Branch, Los Angeles District Corps of Engineers," Contract No. DACW09-87-M-1734.

4. State of California Department of Water Resources (1970) "Meeting Water Demands in the Bunker Hill-San Timoteo Area," Bulletin 104-5.

5. Cameron, C.P., et. al. (1986) "Geotechnical Aspects of Rock Erosion in Emergency Spillway Channels," Technical Report REMR-GT-3, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss.

6. Craemer, C.H., et. al. (1984) "Microearthquake, Geophysical and Geodetic Surveys for Earthquake Hazards Evaluation, Eastern San Gabriel Mountains and Upper Pomona Valley areas, San Bernardino County, California," Open File Report 84-23SAC, California Division of Mines and Geology.

7. Dibblee, T.W., Jr. (1964) "Geologic Map of the San Gorgonio Mountain Quadrangle, San Bernardino and Riverside Counties, California," U.S. Geol. Surv., Misc. Geol. Inv. Map I-431.

8. Dibblee, T.W., Jr. (1968) "Displacements of the San Andreas fault system in the San Gabriel, San Bernardino and San Jacinto Mountains, Southern California," Proceedings, Conference on Geologic Problems of San Andreas Fault System, Stanford University Publication of Geologic Science, v. XI, pp. 260-278.

9. Dibblee, T.W., Jr. (1970) "Regional Geologic Map of San Andreas and related faults in eastern San Gabriel Mountains, San Bernardino Mountains and vicinity," Open File Map, U.S. Geologic Survey, scale 1:125,000.

10. Dibblee, T.W., Jr. (1974) "Geologic Map of the Redlands Quadrangle, California," Open File Map, U.S. Geologic Survey, scale 1:62,500.

11. Dibblee, T.W., Jr. (1975) "Late Quaternary Uplift of the San Bernardino Mountains on the San Andreas and Related Faults," in Crowell, J.C. (ed.), San Andreas Fault in Southern California: Calif. Div. Mines Geol., Special Report 118.

12. Dibblee, T.W., Jr. (1982) "Regional Geology of the Transverse Ranges Province of Southern California," in Fife, D.L. and Minch, J.A. (eds.), Geology and Mineral Wealth of the California Transverse Ranges, Mason Hill Volume: South Coast Geol. Soc.

13. Frizzell, V.A., Jr., Mattinson, J.M., and Matti, J.C. (1986) "Distinctive Triassic Megaporphyritic Monzogranite: Evidence for Only 160 km Offset Along the San Andreas Fault, Southern California," Jour. Geophys. Res., v. 91, no. B14, December.

14. Frost, R.J. (1972) "Some Testing Experiences and Characteristics of Boulder-Gravel Fill in Earth Dams," Evaluation of Relative Density and its Roll in Geotechnical Projects Involving Cohesionless Soils, ASTM STP 523, American Society for Testing and Materials, June 1972, pp. 207-233.

15. Fuscoe, Jenny, Velez (1985) "Preliminary Engineering Report for Outlet Tunnels at Lytle Creek and Upper Santa Ana River Damsites, San Bernardino, California," prepared for U.S. Army Corps of Engineers, Los Angeles District.

16. Humphreys, E. (1986) "The Role of Mantle Involvement in the Recent Development of Southern California," Annual Mtg., Cord. Sec., Geol. Soc. Am. (abstr.).

17. Jacoby, G.C., Sheppard, P.R., and Sieh, K.E. (1987) "Was the 8 December 1812 California Earthquake Produced by the San Andreas Fault? Evidence from Trees near Wrightwood," Seis. Soc. Am., 82nd Annual Mtg. (abstr.).

18. Justin, J.D., Hinds, J., and Craeger, W.P. (1947) Engineering for Dams, Volume III: Earth, Rock-Fill, Steel and Timber Dams, John Wiley and Sons, Inc.

19. Krinitsky, E.L., Chang, F.K. (1975) "State-of-the-Art for Assessing Earthquake Hazards in the United States, Specifying Peak Motions for Design Earthquakes," Miscellaneous Paper S-73-1, Report 7, U.S. Army Engineer Waterways Experiment Station, CE, Vicksburg, MS.

20. Lee, K.L., Focht, J.A. (1975) "Liquefaction Potential at Ekofish Tank in North Sea," Journal, Geotechnical Engineering Division, ASCE, v. 100, GT1.

21. Leps, T.M., Jansen, R.B. (1974) "Proposed Guidelines for Dam Seismic Stability Assessment," USCOLD Newsletter, U.S. Committee on Large Dams, Issue No. 75.

22. Luyendyk, B.P. (1987) "Dextral Shear in Southern California Since Early Miocene Time," Annual Mtg., Geol. Soc. Am. (abstr.).

23. Luyendyk, B.P., Kamerling, M.J., and Terres, R. (1980) "Geometric Model for Neogene Crustal Rotations in Southern California," Geol. Soc. Am. Bull., Part I, v. 91, April.

24. Matti, J.C., Morton, D.M., and Cox, B. (1985) "Distribution and Geologic Relations of Fault Systems in the Vicinity of the Central Transverse Ranges, Southern California," U.S. Geol. Surv., Open File Report 85-365.

25. Meade, R.B., (1982) "The Evidence for Reservoir-Induced Macroearthquakes" Misc. Paper S-73-1, State-of-the-Art for Assessing Earthquake Hazards in the United States, Report 19, Geotechnical Laboratory U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

26. Metzger, L. (1982) "Tectonic implications of the Quaternary history of lower Lytle Creek, Southeastern San Gabriel Mountains," Senior Thesis, Pomona College, Claremont, p. 21.

27. Morton, D.M. (1975) "Synopsis of the Geology of the Eastern San Gabriel Mountains, Southern California," San Andreas Fault in Southern California, (Crowell, J.C., ed.), Special Report 118, California Division of Mines and Geology, pp. 170-176.

28. Morris, H.M. (1963) Applied Hydraulics in Engineering, Ronald Press Co., pp. 490-411.

29. Nicholson, C., Seeber, L., Williams, P., and Sykes, L.R. (1986) "Seismicity and Fault Kinematics Through the Eastern Transverse Ranges, California: Block Rotation, Strike-Slip Faulting and Low-Angle Thrusts," Jour. Geophys. Res., v. 91, no. B5, April.

30. Packer, D.R., Cluff, L.S., Knuepher, P.L., Withers, R.J. (1979) "Study of Reservoir Induced Seismicity," Woodward-Clyde Consultants for U.S. Geologic Survey, Contract No. 14-08-0001-16809.

31. Sadler, P.M., and Reeder, W.A. (1983) "Upper Cenozoic quartzite-bearing gravels of the San Bernardino Mountains, Southern California: recycling and mixing as a result of transpressional uplift," Tectonics and Sedimentation Along Faults of the San Andreas System (Anderson, D.W., and Rymer, M.J., eds.), Pacific Section, Society of Economic Paleontologists and Mineralogists, pp. 45-47.

32. Schaefer, Dixon, Associates (1985) "Geologic Mapping Report, Proposed Upper Santa Ana River Damsite, San Bernardino County, California," prepared for U.S. Army Corps of Engineers, Los Angeles District.

33. Schaefer, Dixon, Associates (1985) "Geologic Mapping Report, Proposed Lytle Creek Damsite, San Bernardino County, California," prepared for U.S. Army Corps of Engineers, Los Angeles District.

34. Schnabel, P.B., Seed, H.B. (1972) "Accelerations in Rock for Earthquakes in the Western United States," Report No. EERC 72-2, Earthquake Engineering Research Center, University of California, Berkeley.

35. Seed, H.B. (1968) "The Fourth Terzaghi Lecture: Landslides During Earthquakes Due to Liquefaction," Journal, Geotechnical Engineering Division, ASCE, v. 94, SM5.

36. Seed, H.B., Arango, I., Chan, C.K. (1975) "Evaluation of Soil Liquefaction Potential During Earthquakes," Report No. EERC 75-28, Earthquake Engineering Research Center, University of California, Berkeley.

37. Sharp, R.P. (1972) "Map showing recently active breaks along the San Jacinto fault zone between the San Bernardino area and Borrego Valley, California," Miscellaneous Geologic Investigations Map I-675, U.S. Geologic Survey, scale 1:24,000.

38. Sieh, K. (1978) "Prehistoric large earthquake produced by slip on the San Andreas fault at Pallett Creek, California," Journal, Geophysical Research, v. 83, pp. 3907-3939.

39. Slemmons, D.B. (1977) "State-of-the-Art for Assessing Earthquake Hazards in the United States, Faults and Earthquake Magnitude," Miscellaneous Paper S-73-1, Report 6, U.S. Army Engineer Waterways Experiment Station, CE, Vicksburg, MS.

40. U.S. Department of the Interior (1974) Earth Manual, U.S. Government Printing Office.

41. Weldon, R.J. and Sieh, K. (1985) "Holocene rate of slip and tentative recurrence interval for large earthquakes on the San Andreas fault in Cajon Pass, Southern California," Geologic Society of America Bulletin, v. 96, pp. 793-812.

42. West, D. L. (1987) "The Geology of the Wilson Creek-Mill Creek Fault Zone - the North Flank of the Former Mill Creek Basin, San Bernardino County, California," M.S. Thesis, University of California, Riverside.

43. Wilson, S. and Marsal, R. (1979) Current Trends in Design and Construction of Embankment Dams, ASCE, prepared for Committee on International Relations, Carlos S. Ospina, Chairman (ICOLD) and Geotechnical Division, ASCE, W. F. Swiger, Chairman.

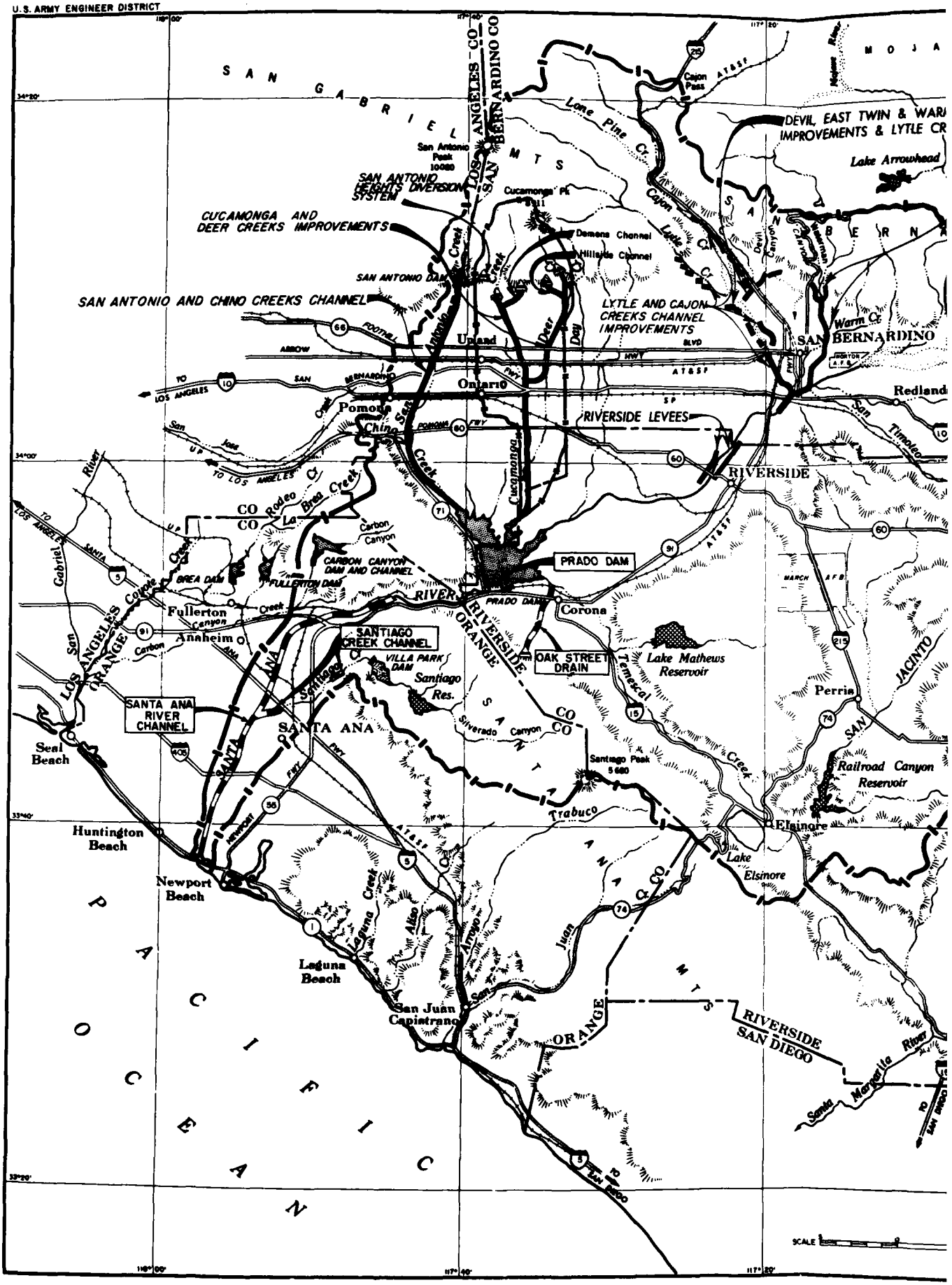
44. Wong, R.T. (1971) "Deformation Characteristics of Gravels and Gravelly Soils Under Cyclic Loading Conditions," Ph.D. Thesis, University of California, Berkeley.

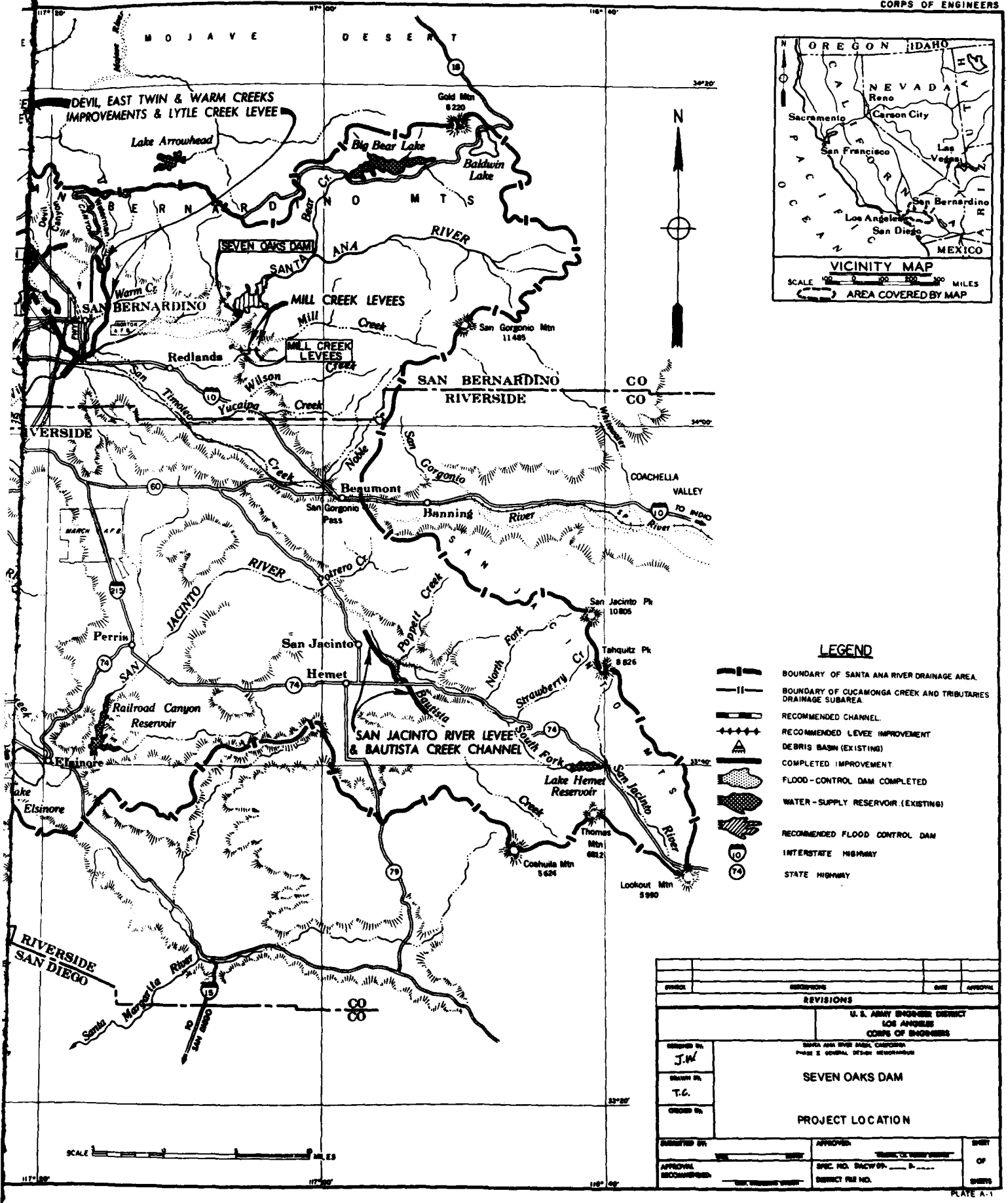
45. Wong, R.T., Seed, H.B., and Chan, C.K. (1974) "Liquefaction of Gravelly Soils Under Cyclic Loading Conditions," Report No. EERC 74-11, Earthquake Engineering Research Center, University of California, Berkeley.

46. Ziony, J.I., ed. (1985) "Evaluating Earthquake Hazards in the Los Angeles Region - An Earth Science Perspective" USGS Prof. Paper 1360.

A GEOTECHNICAL

PLATES


U.S. ARMY ENGINEER DISTRICT
SAN GABRIEL MTS
LOS ANGELES CO
SAN BERNARDINO CO
ORANGE CO
RIVERSIDE CO
SAN DIEGO CO
PACIFIC OCEAN
San Antonio Peak 10080
SAN ANTONIO HEIGHTS DIVERSION SYSTEM
CUCAMONGA AND DEER CREEKS IMPROVEMENTS
SAN ANTONIO AND CHINO CREEKS CHANNEL
LYTLE AND CAJON CREEKS CHANNEL IMPROVEMENTS
DEVIL, EAST TWIN & WARM IMPROVEMENTS & LYTLE CR
RIVERSIDE LEVEES
PRADO DAM
CARBON CANYON DAM AND CHANNEL
SANTIAGO CREEK CHANNEL
OAK STREET DRAIN
SANTA ANA RIVER CHANNEL
Lake Arrowhead
Lake Mathews Reservoir
Railroad Canyon Reservoir
Lake Elsinore
Santiago Peak 5689
Cajon Pass
Lone Pine Cr.
Cucamonga Pk.
Demens Channel
Hillside Channel
Cajon Creek
Lytle Creek
Devil Canyon
Warm Cr.
SAN BERNARDINO
Upland
Ontario
Pomona
Chino
Redlands
RIVERSIDE
Rodeo Cr.
La Brea Creek
Carbon Canyon
BREA DAM
FULLERTON DAM
Fullerton
Anaheim
Santiago Res.
VILLA PARK DAM
Silverado Canyon
Trabuco
Corona
Perris
SANTA ANA
Seal Beach
Huntington Beach
Newport Beach
Laguna Beach
San Juan Capistrano
Laguna Creek
Aliso Creek
Arroyo Trabuco
San Juan Creek
Temescal Creek
Santa Margarita River
MARCH AFB
SCALE



LEGEND

- BOUNDARY OF SANTA ANA RIVER DRAINAGE AREA
- BOUNDARY OF CUCAMONGA CREEK AND TRIBUTARIES DRAINAGE SUBAREA
- RECOMMENDED CHANNEL
- RECOMMENDED LEVEE IMPROVEMENT
- DEBRIS BASIN (EXISTING)
- COMPLETED IMPROVEMENT
- FLOOD-CONTROL DAM COMPLETED
- WATER-SUPPLY RESERVOIR (EXISTING)
- RECOMMENDED FLOOD CONTROL DAM
- INTERSTATE HIGHWAY
- STATE HIGHWAY

MARK	DESCRIPTION	DATE	APPROVAL
	REVISIONS		

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

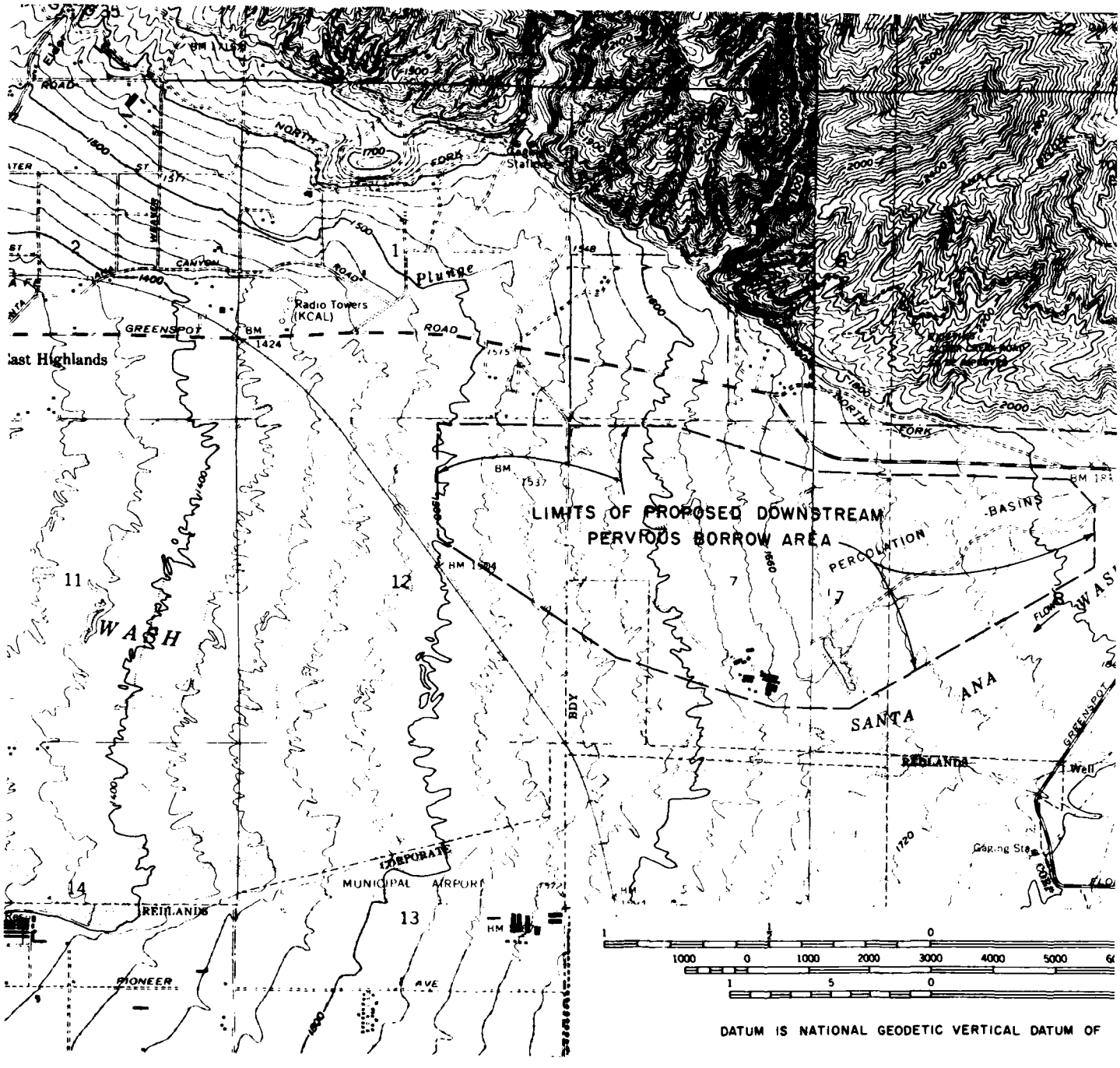
SANTA ANA RIVER BASIN, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

PROJECT LOCATION

DESIGNED BY: J.W.
DRAWN BY: T.C.
CHECKED BY:
SUBMITTED BY:
APPROVED:
APPROVAL RECOMMENDED:
SPEC. NO. DACW09-
DISTRICT FILE NO.
SHEET OF SHEETS

PLATE A-1



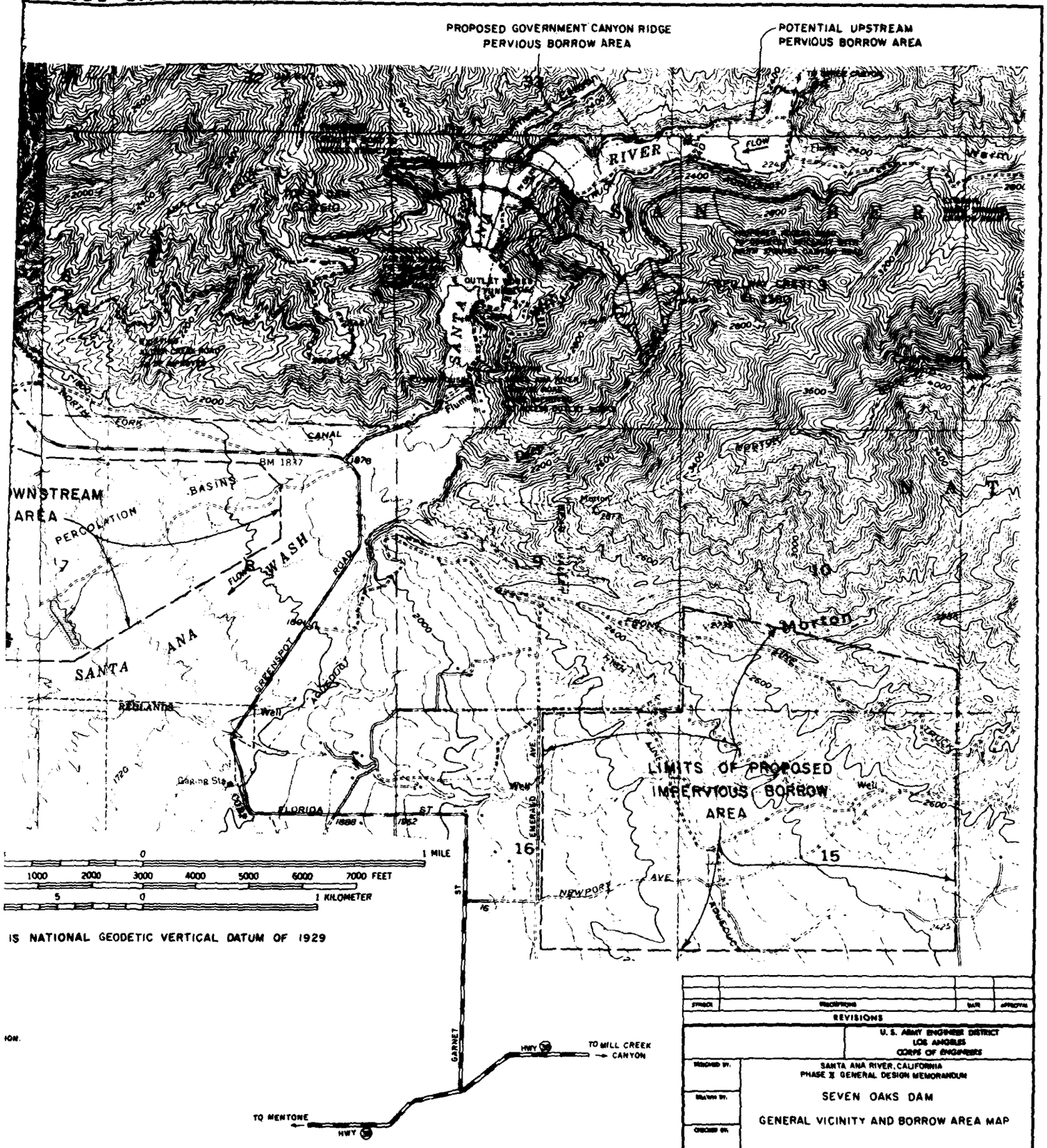
NOTES:

1. SEE PLATE A-9 FOR PLAN OF EXPLORATION AT DAMSITE AND SPILLWAY.
2. SEE PLATE A-55 FOR DOWNSTREAM PERVIOUS BORROW AREA PLAN OF EXPLORATION.
3. SEE PLATE A-60 FOR IMPERVIOUS BORROW AREA PLAN OF EXPLORATION.

TO MENTO

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

VALUE ENGINEERING PAYS



SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

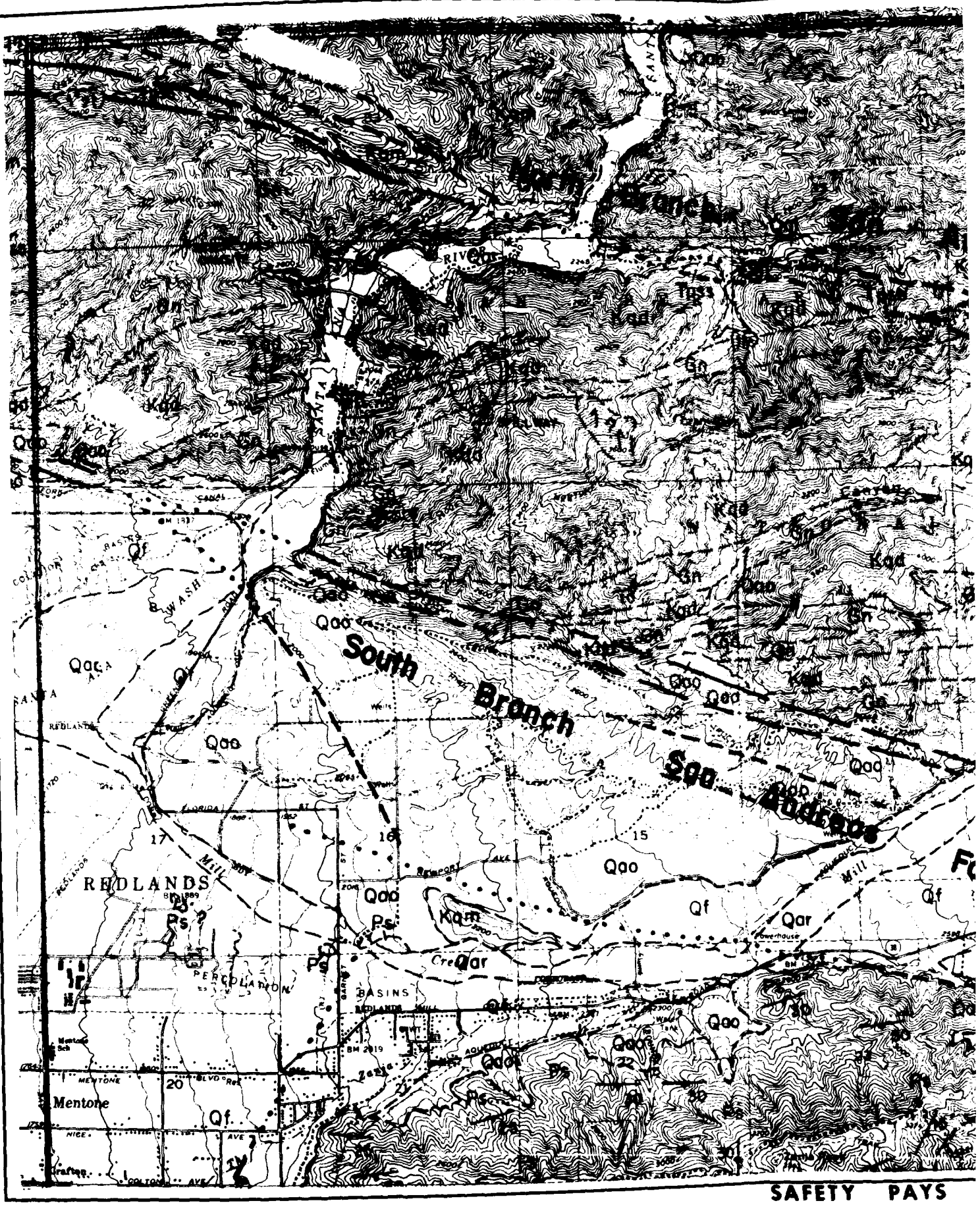
GENERAL VICINITY AND BORROW AREA MAP

SUBMITTED BY: DATE APPROVED: SPEC. NO. DACW09-____-____ SHEET

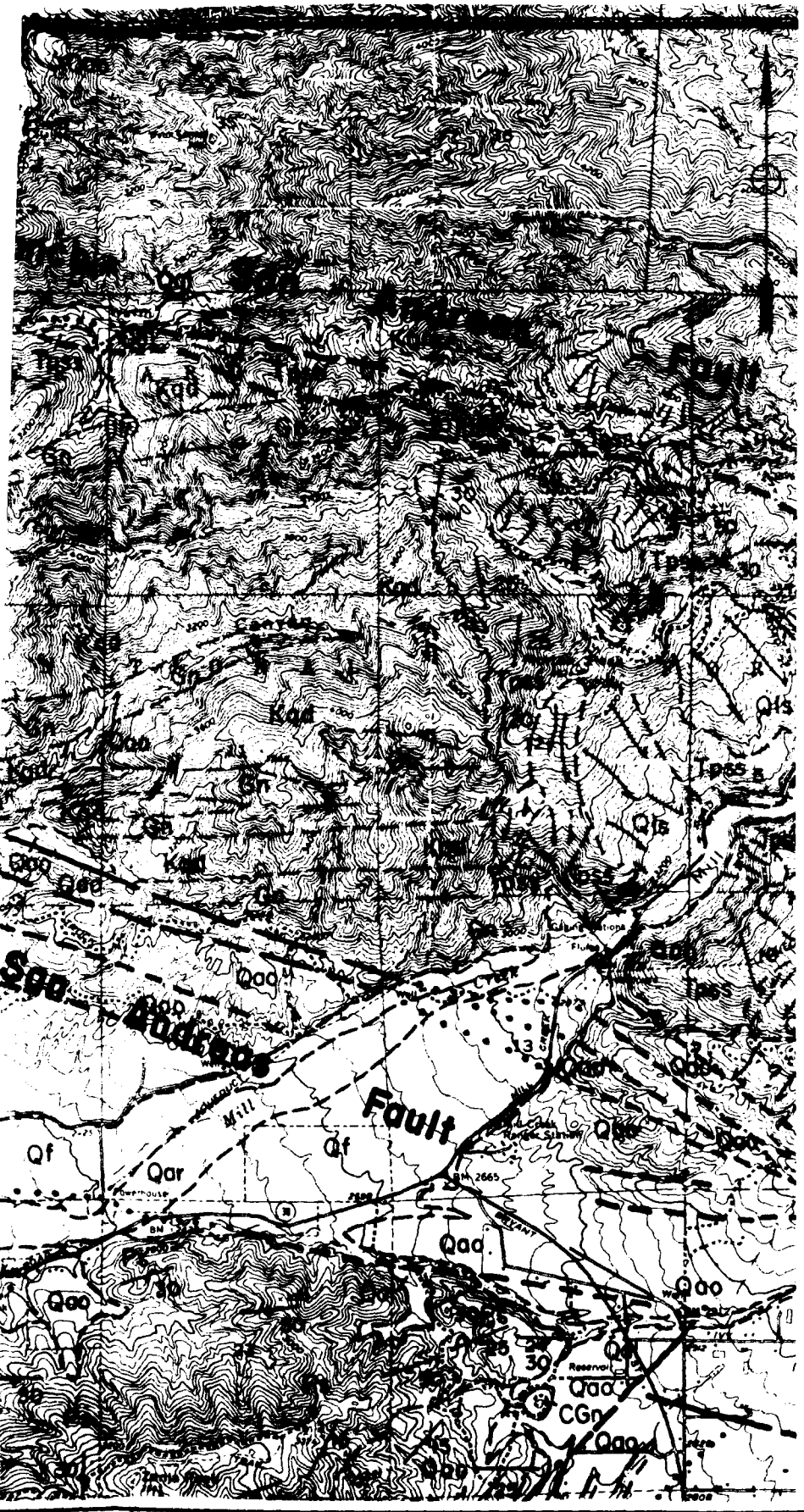
DISTRICT FILE NO.

SAFETY PAYS

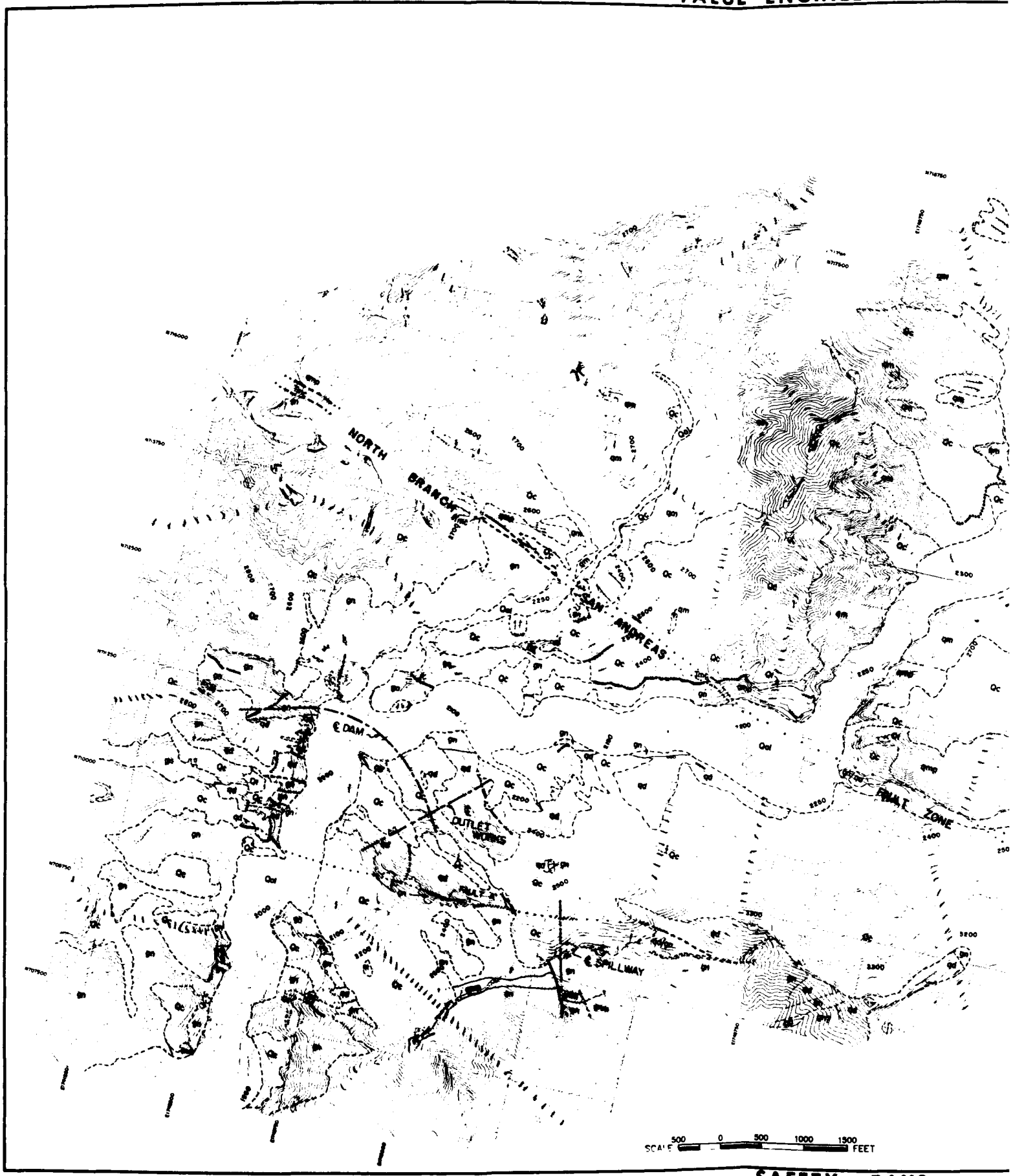
PLATE A-2



ENVIRONMENTAL
ENHANCEMENT
THRU ENGINEERING

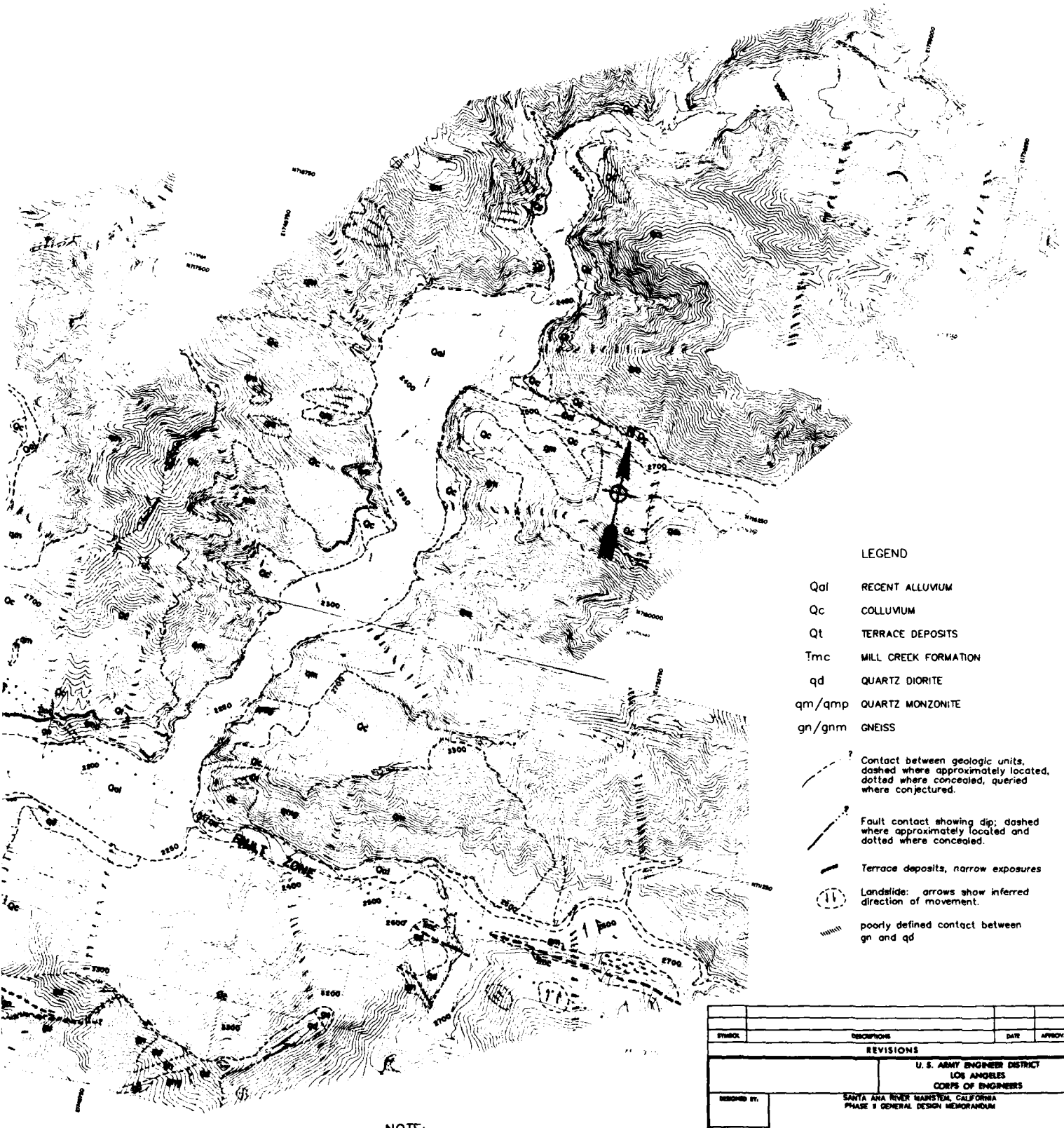

Qaa
Fault
Qaa
Qaa
Qaa

VALUE ENGINEERING PAYS



SAFETY PAYS

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING



LEGEND

Qal	RECENT ALLUVIUM
Qc	COLLUVIUM
Qt	TERRACE DEPOSITS
Tmc	MILL CREEK FORMATION
qd	QUARTZ DIORITE
qm/qmp	QUARTZ MONZONITE
gn/gnm	GNEISS

Contact between geologic units, dashed where approximately located, dotted where concealed, queried where conjectured.

Fault contact showing dip; dashed where approximately located and dotted where concealed.

Terrace deposits, narrow exposures

Landslide: arrows show inferred direction of movement.

poorly defined contact between gn and qd

NOTE:

1. See plate A-6 for detailed description of geologic units.

500 0 500 1000 1500 FEET

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

LOCAL GEOLOGY

SUBMITTED BY: DATE APPROVED:

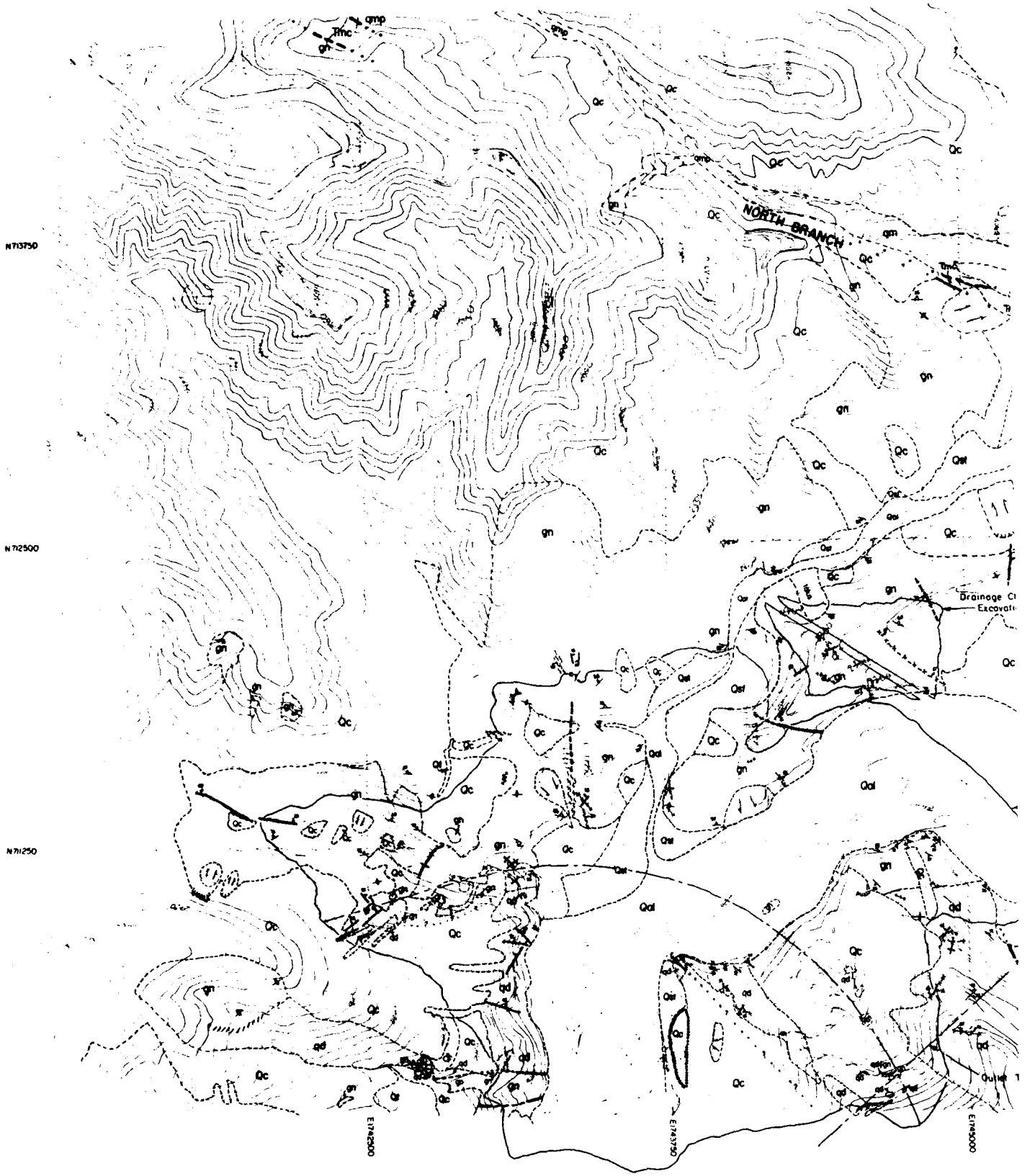
CHIEF, SURVEY

SPEC. NO. DACW09-____-B-____

DISTRICT FILE NO.

SHEET

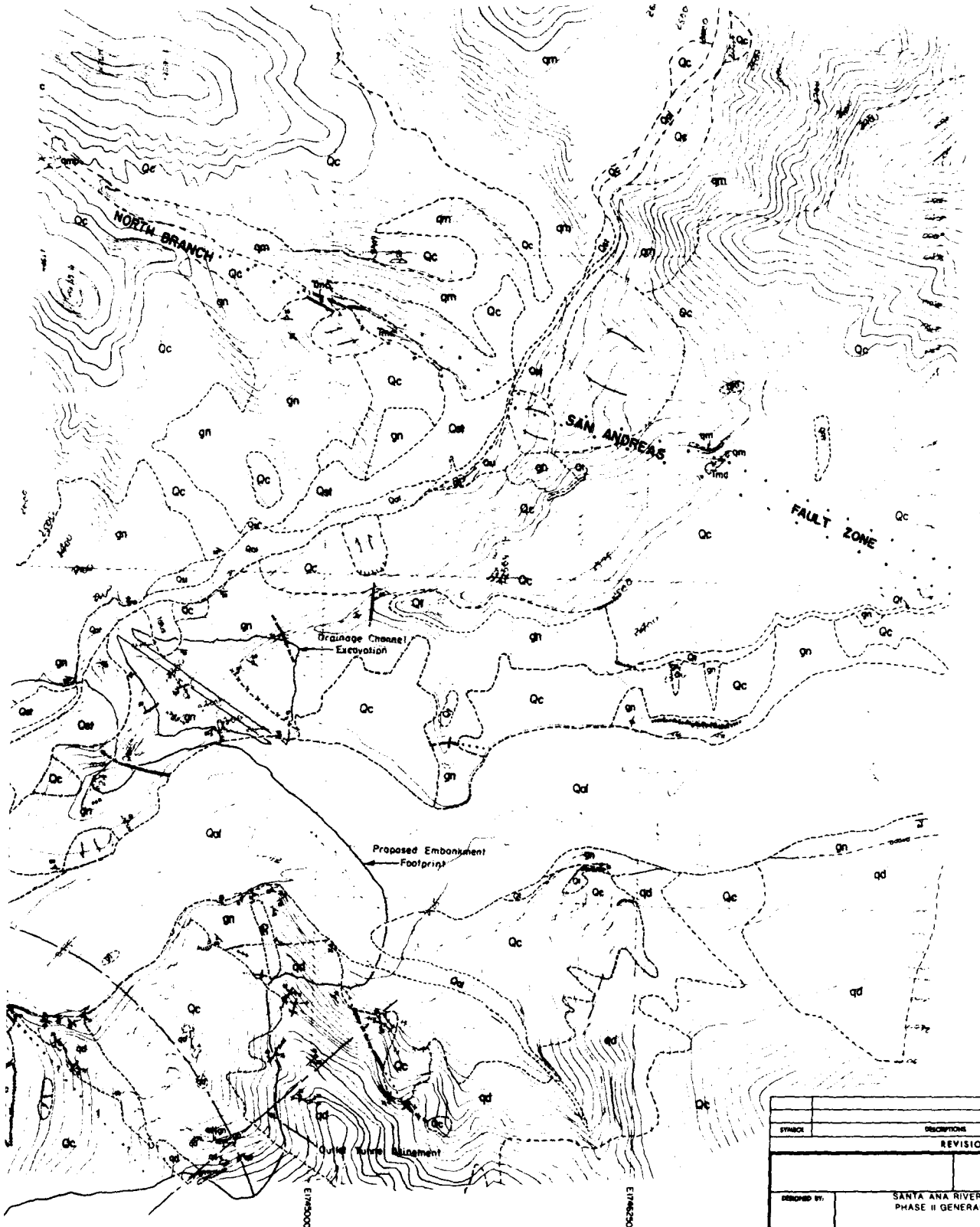
PLATE A-4



NOTE:

See Plate A-6 for legend of geologic symbols.

ENVIRONMENTAL
ENHANCEMENT
THRU ENGINEERING



SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

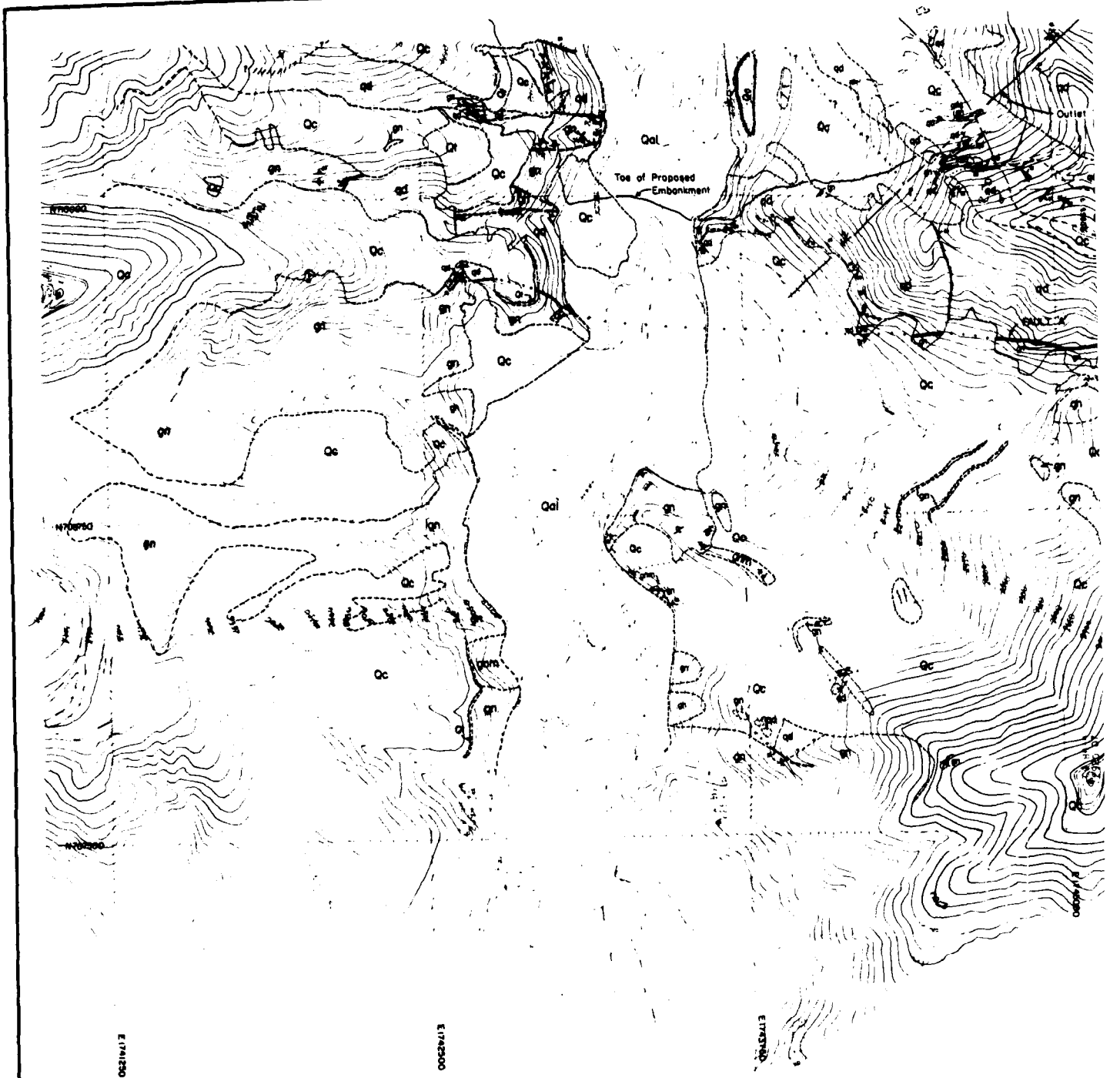
SITE GEOLOGY

SUBMITTED BY: DATE APPROVED:
CHIEF, BRANCH

SPEC. NO. DACW09-____-B-____
DISTRICT FILE NO.

SHEET 1 OF 2 SHEETS

PLATE A-6



ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

Terrace deposits: shown for narrow exposures.

Strike and dip of joint.

Strike of vertical joint.

Strike and dip of foliation.

Strike of vertical foliation.

Poorly defined contact between gn and gd.

Contact between geologic units, dashed where approximately located, dotted where concealed, queried where conjectured.

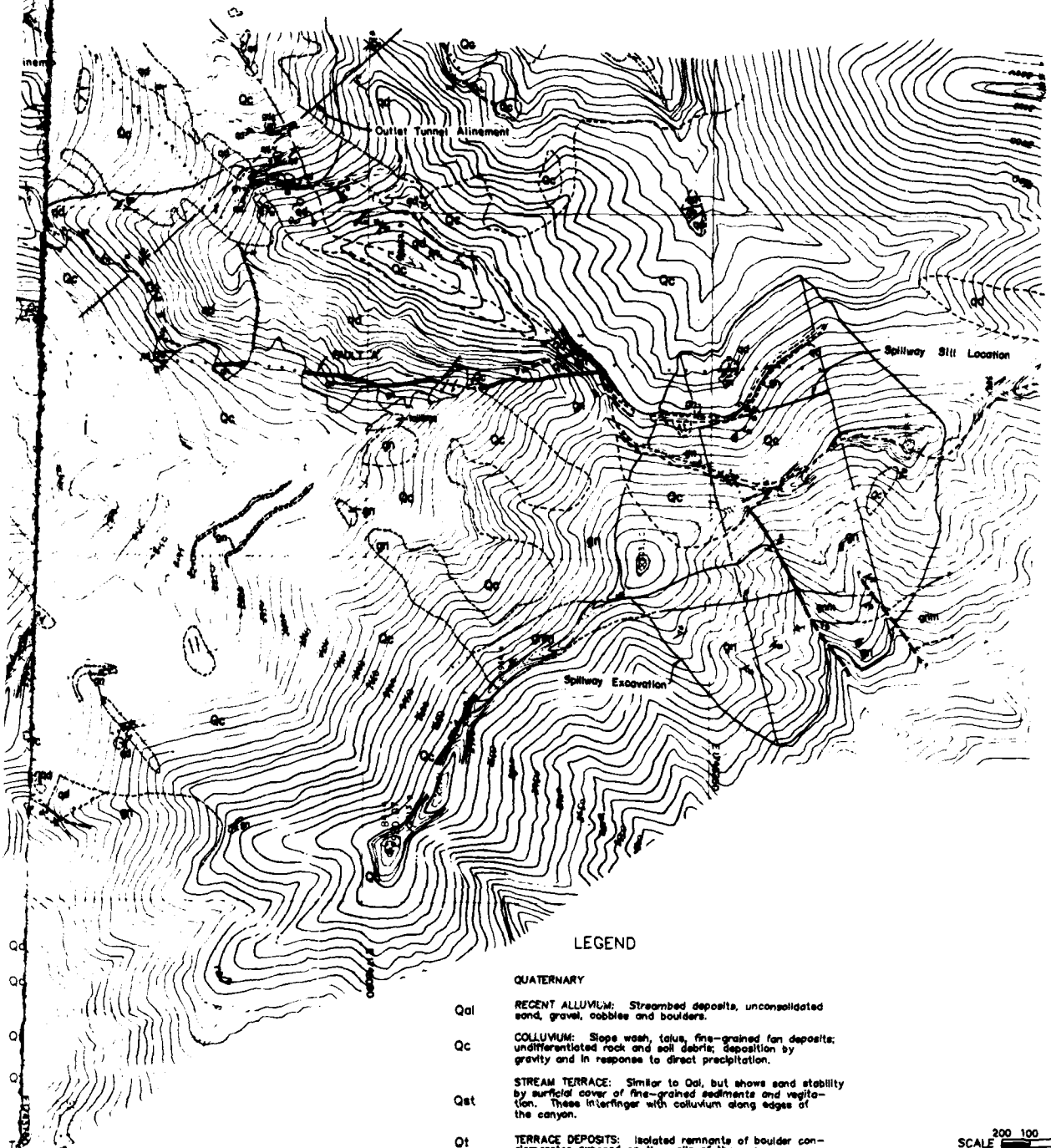
Fault contact showing dip; dashed where approximately located, dotted where concealed and queried where conjectured.

DIKE: Cretaceous(?); cuts diorite and gneiss; all are aplite dikes.

Landslide scarp: hachures on down side.

Landslide: arrows show inferred direction of movement.

VALUE ENGINEERING PAYS



LEGEND

QUATERNARY

Qal — RECENT ALLUVIUM: Streambed deposits, unconsolidated sand, gravel, cobbles and boulders.

Qc — COLLUVIUM: Slope wash, talus, fine-grained fan deposits; undifferentiated rock and soil debris; deposition by gravity and in response to direct precipitation.

Qst — STREAM TERRACE: Similar to Qal, but shows sand stability by surficial cover of fine-grained sediments and vegitation. These interfinger with colluvium along edges of the canyon.

Qt — TERRACE DEPOSITS: Isolated remnants of boulder conglomerates exposed on the walls of the canyon.

TERTIARY

Tmc — Mill Creek formation: moderately to well indurated terrestrial and lucustrine conglomerate, sandstone and shale.

CRETACEOUS ~ JURASSIC

qd — QUARTZ DIORITE: Grey, massive, medium-grained crystals, unweathered to slight weathering of feldspar and mafic minerals, typically highly fractured, locally sheared and altered.

qm, qmp — QUARTZ MONZONITE: Equigranular, occasional xenoliths of mafic minerals, moderately jointed; qmp: contains large phenocrysts of potassic feldspar up to 3-inch diameter.

PRECAMBRIAN:

gn, gnm — GNEISS: Undifferentiated, dioritic, grey to black, fine-grained crystals, massive to foliated along subparallel mafic zones, principal constituents are plagioclase and potassic feldspar, quartz and mafic minerals, moderate to complete weathering of the feldspar and mafic minerals; gnm: zone of gneiss with mafic mineral content of rock greater than 25 percent.

...ntact between geologic units, dashed where approximately ...ated, dotted where concealed, queried where conjectured.

...lt contact showing dip; dashed where approximately located, ...ted where concealed and queried where conjectured.

...KE: Cretaceous(?); cuts diorite and gneiss; all are aplite dikes.

...ndslide scarp: hachures on down side.

...ndslide: arrows show inferred direction of movement.

SCALE 200 100 0 200 400 600 FEET

SYMBOL	DESCRIPTION	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE I GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

SITE GEOLOGY

SUBMITTED BY: DATE APPROVED:

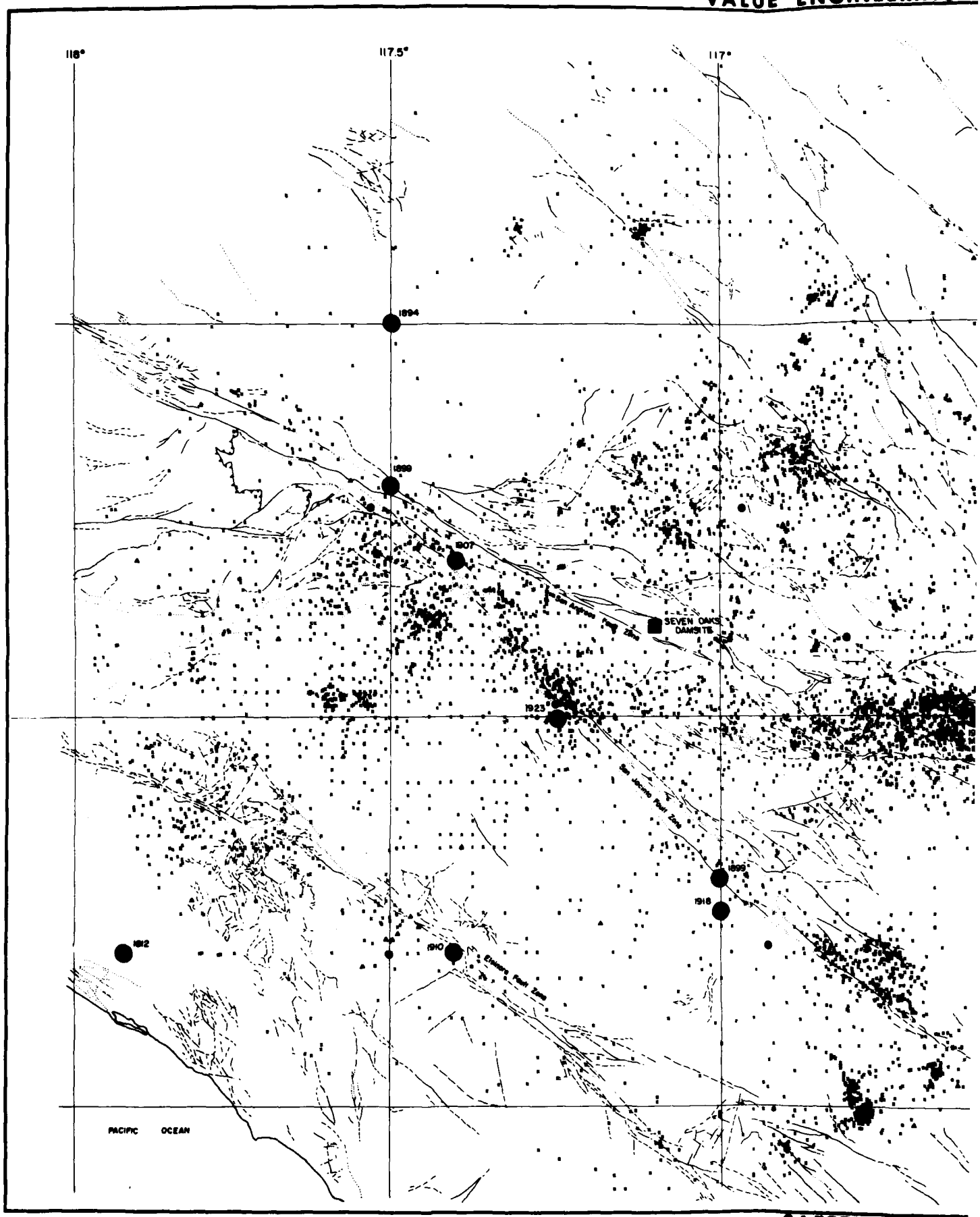
SPEC. NO. DACW09-____-____

DISTRICT FILE NO.

SHEET 2 OF 2 SHEETS

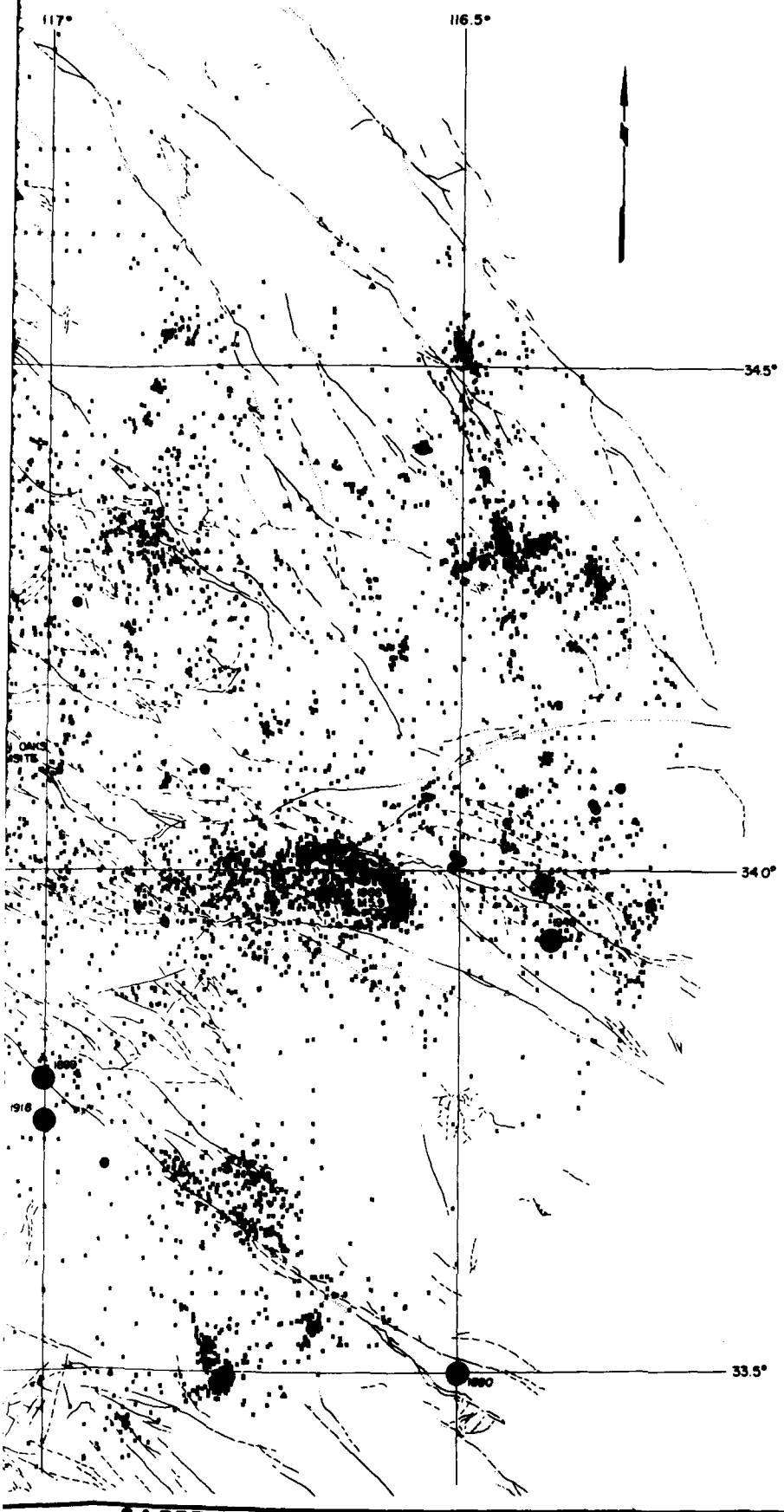
PLATE A-6

SAFETY PAYS



ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

VALUE ENGINEERING PAYS



LEGEND

- GENERAL PROJECT SITE LOCATION
- EARTHQUAKE EPICENTER - MAGNITUDE 2.0-3.9
- MAGNITUDE 4.0-4.9
- MAGNITUDE 5.0-5.9
- 1948 MAGNITUDE 6+ EARTHQUAKE IN 1948
- FAULT - dashed where approximately located, dotted where concealed
- THRUST FAULT

GENERAL NOTES

1. All earthquakes with magnitudes 2.0 or greater from 1932 to 1967 within a 50-mile radius are plotted.
2. Base fault map compiled from the CDMG Geologic Map of California.
3. The 1986 magnitude 5.9 North Palm Springs Earthquake is plotted due to its recency, magnitude and proximity.
4. Historic earthquakes with magnitudes 6.0 or greater are shown with year of occurrence.

SCALE 5 0 5 10 15 20 MILES

SYMBOL	DESCRIPTION	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

FAULT AND EARTHQUAKE EPICENTER MAP

SUBMITTED BY: DATE APPROVED:

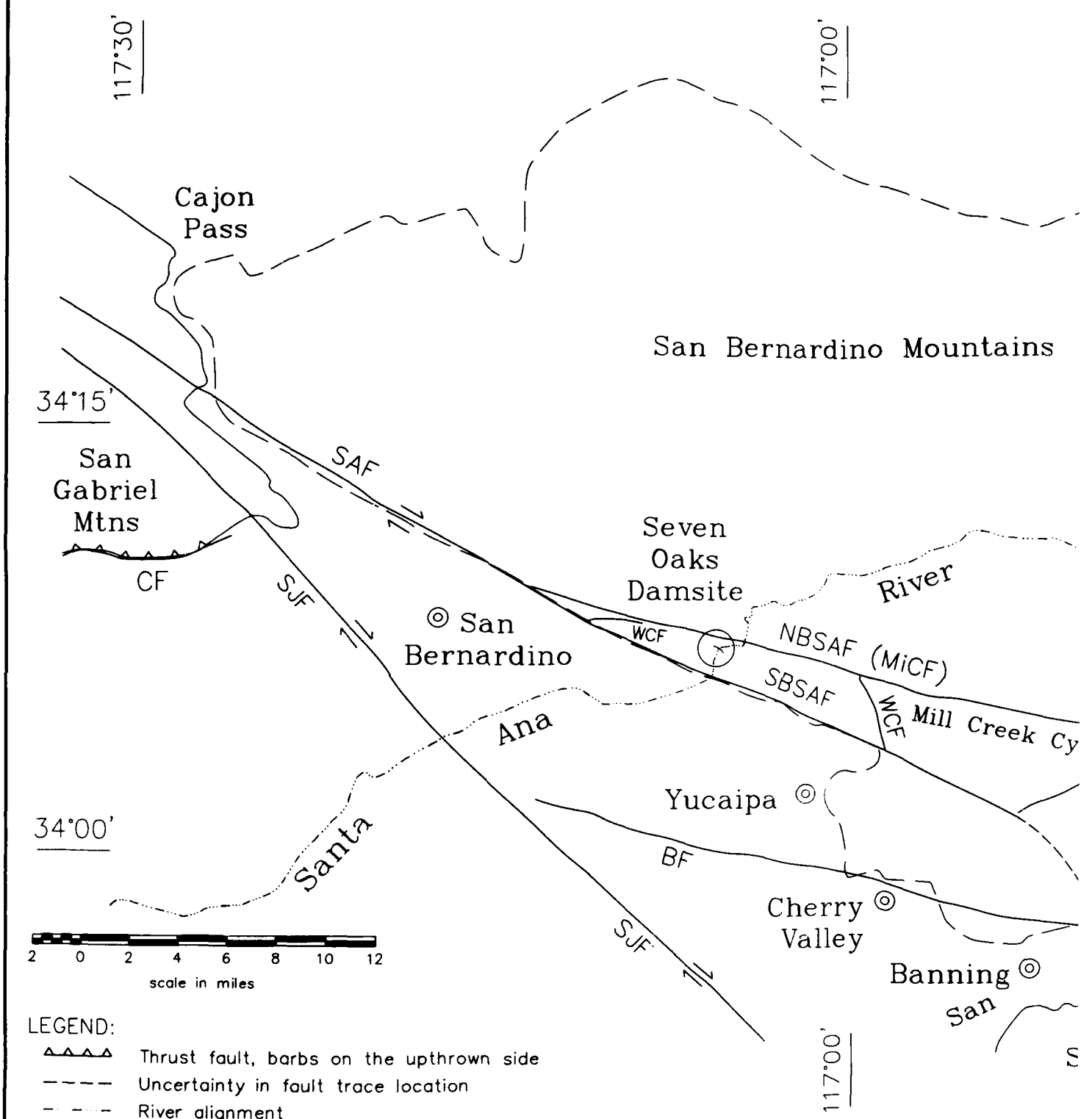
SPEC. NO. DACW09-___-___

DISTRICT FILE NO.

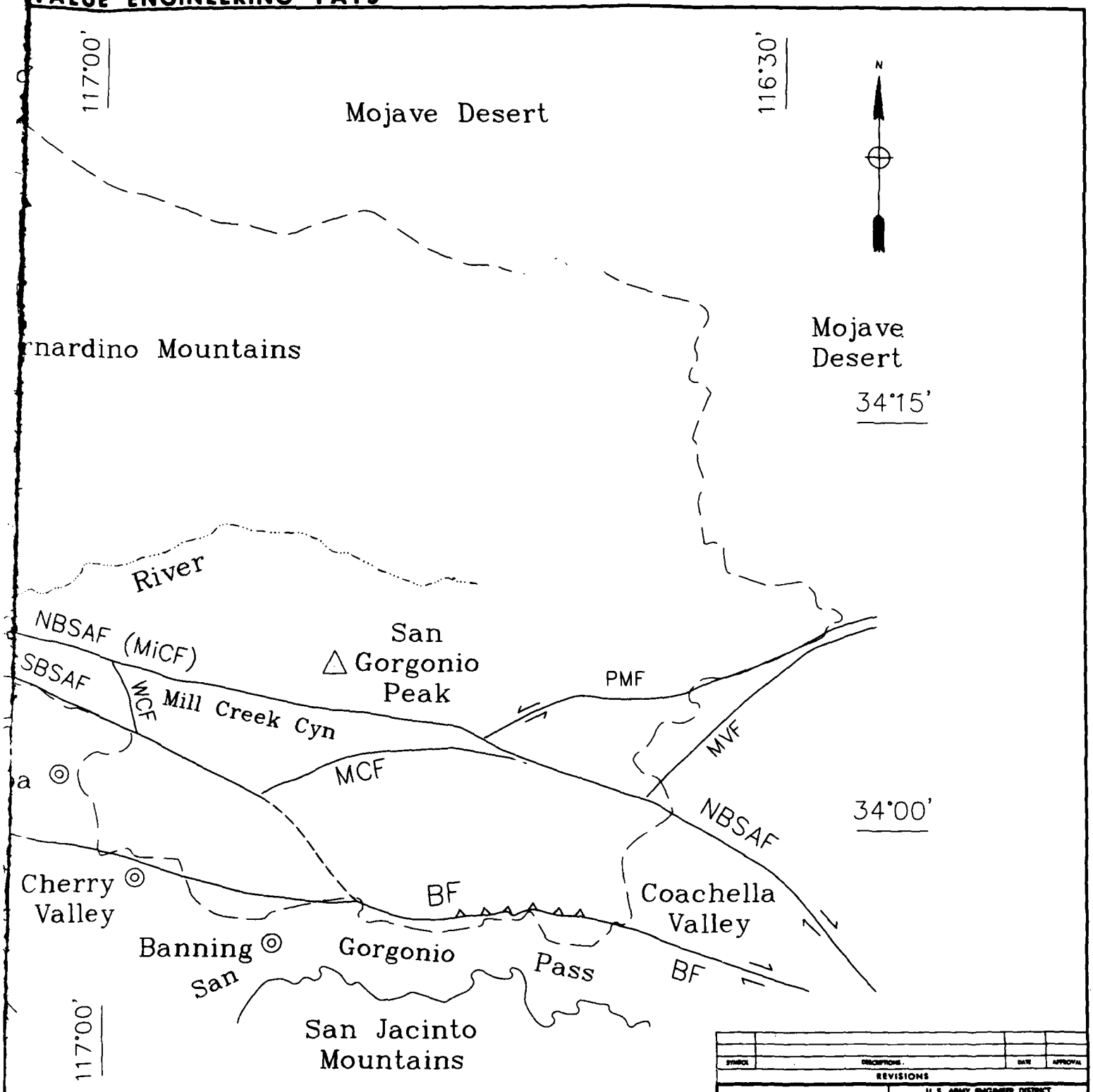
SHEET

PLATE A-7

SAFETY PAYS



ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING



South Branch, San Andreas Fault
San Jacinto Fault
Wilson Creek Fault

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

REGIONAL TECTONIC MAP

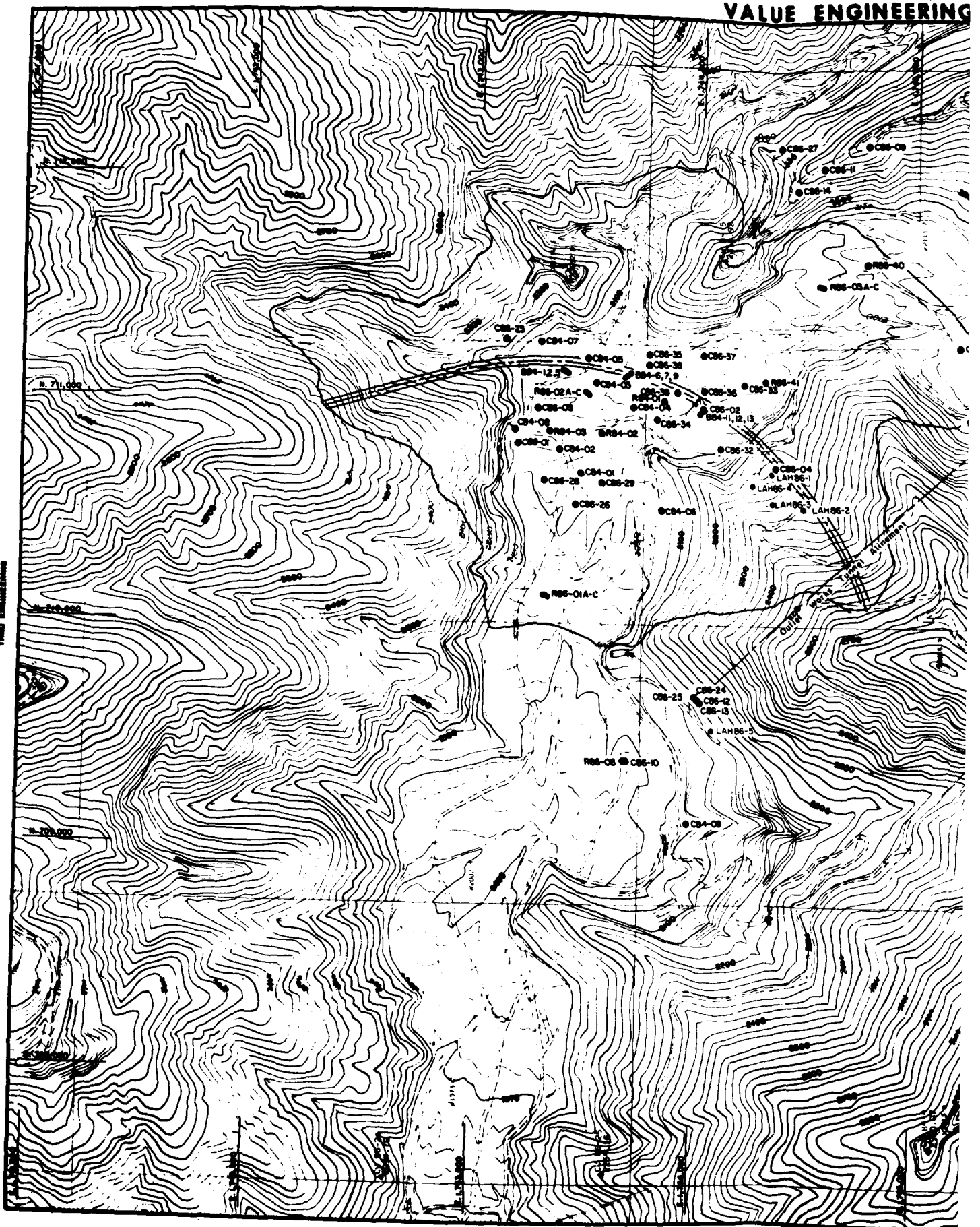
SUBMITTED BY: DATE APPROVED:

SPEC. NO. DACW09-___-B-___

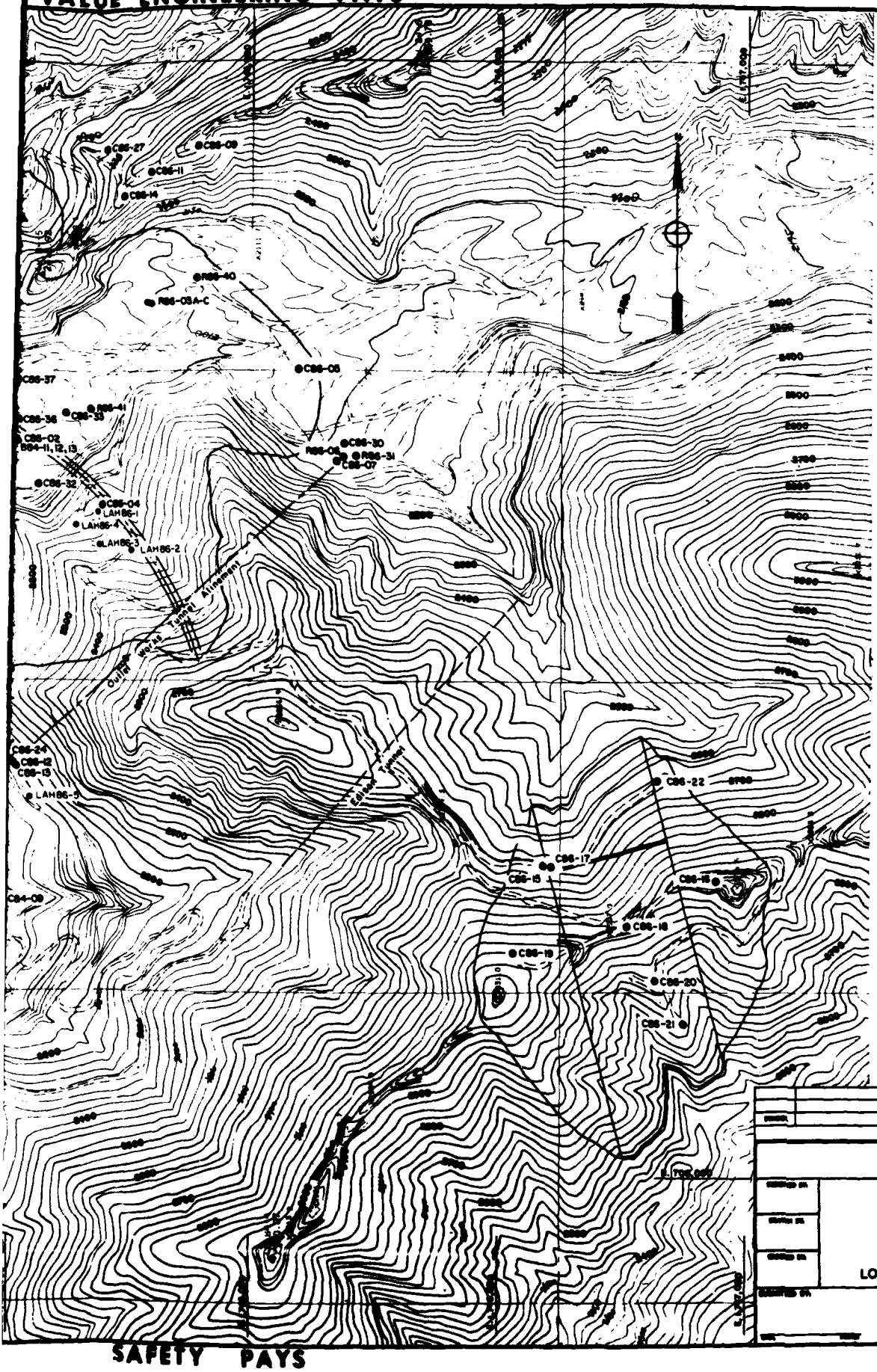
DISTRICT FILE NO.

SHEET

PLATE X-8


CB6-27
CB6-09
CB6-11
CB6-14
RB6-40
RB6-03A-C
CB6-23
CB4-07
CB4-05
CB6-35
CB6-38
CB6-37
BB4-12,5
BB4-6,7,9
CB4-03
RB6-02A-C
CB6-39
CB6-36
CB6-33
RB6-41
CB6-02
BB4-11,12,13
CB4-04
CB6-34
CB4-08
RB4-03
BR84-02
CB6-01
CB4-02
CB6-32
CB6-04
LAH86-1
CB4-01
CB6-28
CB6-29
LAH86-4
LAH86-3
LAH86-2
CB6-26
CB4-06
Tunnel Alignment
RB6-01A-C
Outlet Works
CB6-25
CB6-24
CB6-12
CB6-13
LAH86-5
RB6-08
CB6-10
CB4-09

VALUE ENGINEERING PAYS



LEGEND

● C86-16	Diamond core hole
● RB4-01	Rotary-wash hole
● B84-05	Becker-hammer hole
● RB6-01A-C	Rotary-drilled crosshole set
● LAH86-3	Bucket auger hole

NOTE:

1. See plates A-11 thru A-29 for Drill Logs of exploratory diamond core and rotary-wash holes.

SCALE: 1 IN. = 200 FT.

100 0 100 200 300 400 500

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY
DRAWN BY
CHECKED BY

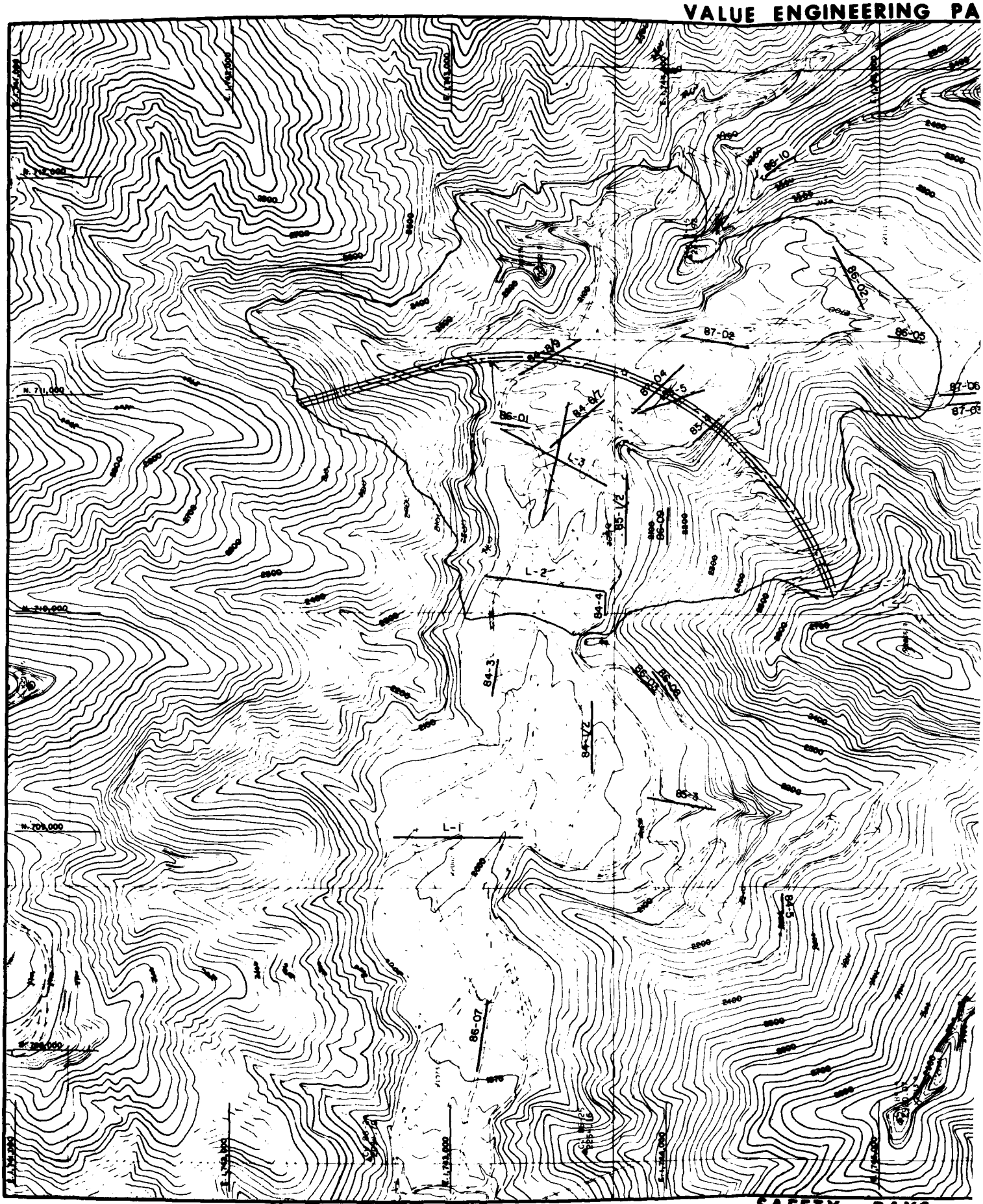
SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM
DAMSITE
PLAN OF EXPLORATION
LOCATIONS OF EXPLORATORY HOLES

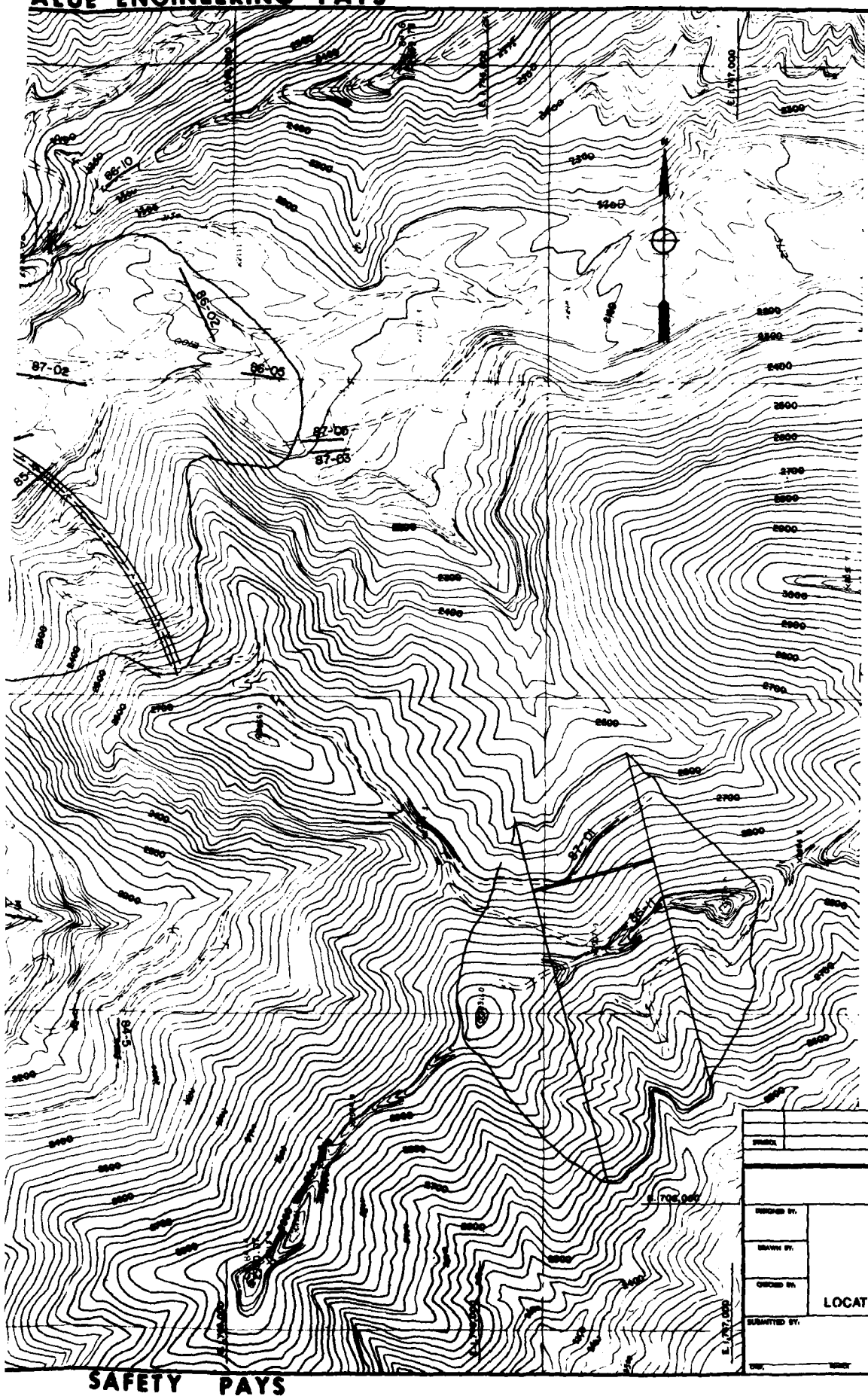
SUBMITTED BY | DATE SUBMITTED | SPEC. NO. SWD/WR ____ S ____ | SHEET
DIVISION | BRANCH | DISTRICT FILE NO.

PLATE A-9

SAFETY PAYS


86-10
86-02
87-02
86-05
87-06
87-03
86-01
84-4/5
L-3
85-1/2
86-09
L-2
84-4
84-3
86-08
86-03
84-2/3
86-4
L-1
86-07
84-5
N. 712,000
N. 710,000
N. 709,000
N. 708,000
E. 1,742,500
E. 1,743,000
E. 1,743,500
E. 1,744,000
E. 1,744,500
E. 1,745,000
E. 1,745,500
E. 1,746,000
E. 1,746,500
E. 1,747,000
E. 1,747,500
E. 1,748,000
E. 1,748,500
E. 1,749,000
E. 1,749,500
E. 1,750,000
E. 1,750,500
E. 1,751,000
E. 1,751,500
E. 1,752,000
E.

ALUE ENGINEERING PAYS



NOTES:

1. See plates A-36 through A-39 for traveltime curves and interpretations.
2. Where two separate lines with different geophone spacings were run at the same location, the line number is represented with a slash (84-1/2)
3. Seismic lines L-1 through L-4 were conducted by Leighton and Associates, see attachment A-4.

SCALE: 1 IN. = 200 FT.

100 0 100 200 300 400 500

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM
DAMSITE
PLAN OF EXPLORATION
LOCATIONS OF SEISMIC REFRACTION LINES

SUBMITTED BY: DATE APPROVED:
CHIEF, BRANCH

SPEC. NO. DACW09-____-____
DISTRICT FILE NO.

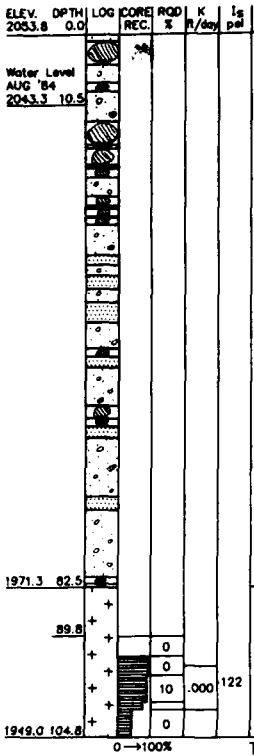
SHEET

SAFETY PAYS

PLATE A-10

C84-01

N 710667.1
E 1743459.2
INCLINATION: VERTICAL



DESCRIPTION

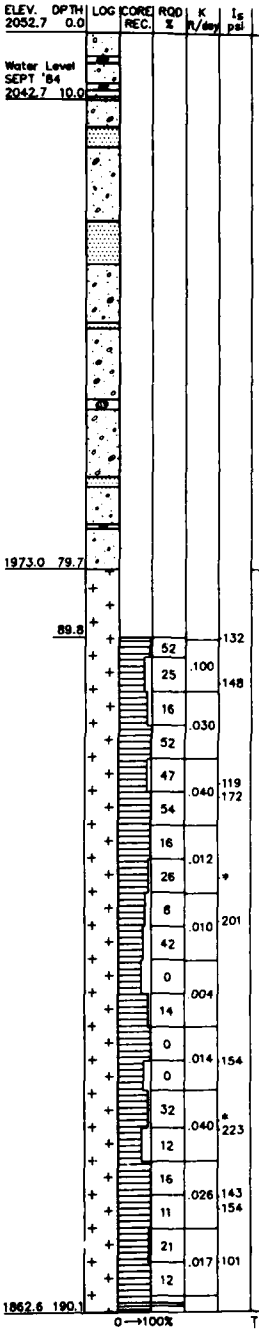
Qal: alluvium; unconsolidated sand, gravel, cobbles, and boulders to 3.5-foot maximum diameter; slightly clayey from 59-60.5' and 71-81'.

qd: diorite; speckled light greyish-white and dark grey to black; medium grained; moderately to completely weathered; generally crumbly core, but locally moderately hard; intensely fractured at random orientations; fracture surfaces smooth to rough, many with slickensides; no samples lab tested.

Total core recovery for interval cored = 72%
Composite RQD = 3%
Average discontinuities/ft = 9.4

C84-02

N 710772.4
E 1743362.7
INCLINATION = 90°



DESCRIPTION

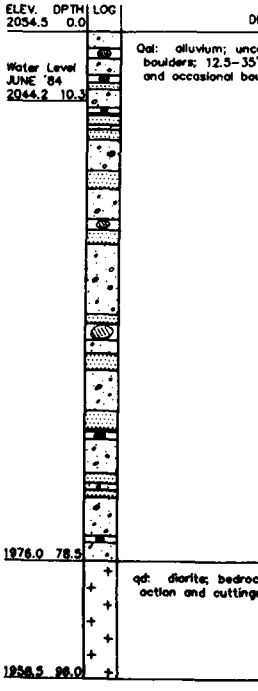
Qal: alluvium; unconsolidated sand, gravel, cobbles, and boulders to 1.5-foot maximum diameter.

Kqd: diorite; speckled light greyish-white and dark grey to black; medium grained; moderately weathered and moderately hard to completely weathered and crumbly from 0-156', generally slightly to moderately weathered and moderately hard to hard, locally highly weathered from 156-190.1'; highly to intensely fractured at random orientations; fracture surfaces smooth to rough, occasionally healed with thin clay gouge, some with slickensides; shear zones dipping 60-70 degrees and containing moderately hard gouge from 104-104.3', 115.7-117.7', and 144.5-144.9'; possible joint sets dip 10-20, 40-50, and 60-70 degrees.

Total core recovery for interval cored = 88%
Composite RQD = 22%
Average discontinuities/ft = 7.3

R84-03

N 710857.0
E 1743314.8
INCLINATION: VERTICAL



DESCRIPTION

Qal: alluvium; unconsolidated sand, gravel, cobbles, and boulders; 12.5-35' slightly clayey sand, gravel, cobbles, and occasional boulders.

qd: diorite; bedrock identification based upon drilling action and cuttings.

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

LUE ENGINEERING PAYS

-02
772.4
3362.7
ATION = 90°

DESCRIPTION

luvium; unconsolidated sand, gravel, cobbles, and lers to 1.5-foot maximum diameter.

diorite; speckled light greyish-white and dark grey lack; medium grained; moderately weathered and moder- y hard to completely weathered and crumbly from 0-156', rally slightly to moderately weathered and moderately l to hard, locally highly weathered from 156-190.1'; ly to intensely fractured at random orientations; ture surfaces smooth to rough, occasionally healed thin clay gouge, some with slickensides; shear zones ing 60-70 degrees and containing moderately hard gouge n 104-104.3', 115.7-117.7', and 144.5-144.9'; possible t sets dip 10-20, 40-50, and 60-70 degrees.

core recovery for interval cored = 88%
osite RQD = 22%
ge discontinuities/ft = 7.3

R84-01

N 710998.4
E 1743831.4
INCLINATION: VERTICAL

ELEV.	DPTH	LOG	DESCRIPTION
2065.0	0.0		
Water Level JUNE '84			
2059.0	6.0		Qal: alluvium; unconsolidated sand, gravel, cobbles, and occasional boulders; 30-49.6' slightly clayey sand, gravel, cobbles, and occasional boulders.
2015.4	49.6		qd: diorite; bedrock identification based upon drilling action and cuttings.
1995.0	70.0		

R84-02

N 710848.1
E 1743550.0
INCLINATION: VERTICAL

ELEV.	DPTH	LOG	DESCRIPTION
2057.8	0.0		
Water Level JUNE '84			
2049.4	8.4		Qal: alluvium; unconsolidated sand, gravel, cobbles, and occasional boulders to 5.5-foot maximum diameter.
2011.3	46.5		qd: diorite; bedrock identification based upon drilling action and cuttings.
1990.3	67.5		

LITHOLOGIC LEGEND

Overburden:

Qal	Alluvium
Qc	Colluvium
	Sand
	Boulder

Bedrock:

qd	Diorite
gn	gneiss

NOTE:

1. See plate A-12 for GENERAL NOTES relating to drill logs.

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

DRILL LOGS
STREAMBOTTOM

SUBMITTED BY: DATE APPROVED:
CHIEF, BRANCH

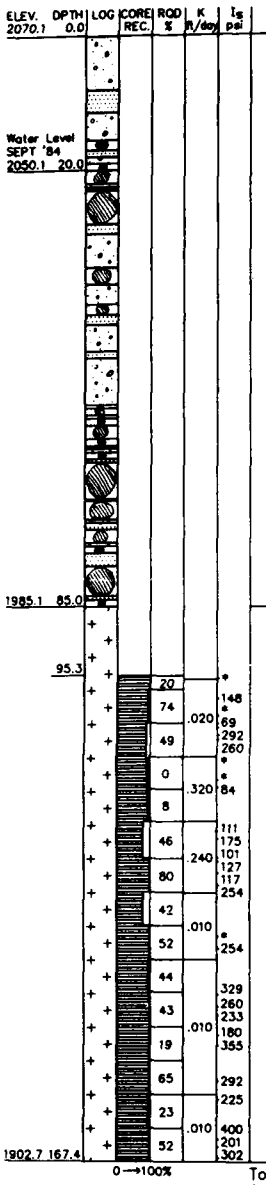
SPEC. NO. DACW 09- - B-
DISTRICT FILE NO.

SHEET 1 OF 7 SHEETS
PLATE A-11

SAFETY PAYS

C84-03

N 711072.2
E 1743521.6
INCLINATION: VERTICAL



DESCRIPTION

Qal: alluvium; unconsolidated sand, gravel, cobbles, and boulders to 5.5-foot maximum diameter.

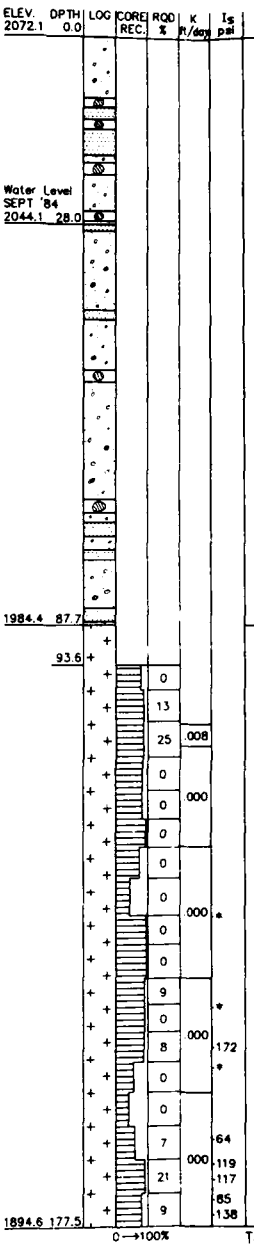
qd: diorite; speckled light greyish-white and dark grey to black; medium grained; generally moderately weathered and moderately hard from 95.3-117', slightly to moderately weathered and moderately hard to hard from 117-167.4'; highly sheared at random orientations; fracture surfaces smooth to rough, occasionally healed or containing thin clay gouge, some with slickensides; possible joint sets dip 40-50 and 60-70 degrees; one sample tested for compressive strength; see general note 4.

RQD %	K ft/day	Is psi
20		*
74	.020	148
49		69
		292
		260
0		*
	.320	*
8		84
		111
46		175
	.240	101
80		127
		117
		254
42	.010	
52		*
		254
44		329
		260
43		233
	.010	180
19		355
65		292
		225
23	.010	400
52		201
		302

Total core recovery for interval cored = 94%
Composite RQD = 42%
Average discontinuities/ft = 5.4

C84-05

N 711181.5
E 1743483.8
INCLINATION: VERTICAL



DESCRIPTION

Qal: alluvium; unconsolidated sand, gravel, cobb
occasional boulders to 2-foot maximum diamet
clayey from 47-49.7' and 71-71.8'.

qd: diorite, speckled light greyish-white and d
to black or greenish-black; medium grained; n
weathered and moderately hard to completely
and soft; intensely fractured at random orient
fracture surfaces generally smooth but locally
some fractures healed by quartz veins with as
potassic alteration or contain minor calcite; m
contain clay gouge, some with slickensides; po
joint sets dip 40-50 and 60-70 degrees; no
tested.

RQD %	K ft/day	Is psi
0		
13		
25	.008	
0		
0	.000	
0		
0		
0	.000	*
0		
0		
9		*
0	.000	
8		172
0		*
0		
7		64
21	.000	119
		117
9		85
		138

Total core recovery for interval cored
Composite RQD = 5%
Average discontinuities/ft = 7.5

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

C84-05

N 711181.5
E 1743483.8
INCLINATION: VERTICAL

RQD %	K ft/day	I_s psi	DESCRIPTION
			Qal: alluvium; unconsolidated sand, gravel, cobbles, and occasional boulders to 2-foot maximum diameter; slightly clayey from 47-49.7' and 71-71.8'.
			qd: diorite; speckled light greyish-white and dark grey to black or greenish-black; medium grained; moderately weathered and moderately hard to completely weathered and soft; intensely fractured at random orientations; fracture surfaces generally smooth but locally rough; some fractures healed by quartz veins with associated potassic alteration or contain minor calcite; many shears contain clay gouge, some with slickensides; possible joint sets dip 40-50 and 60-70 degrees; no samples lab tested.
0			
13			
25	.008		
0			
0	.000		
0			
0			
0			
0	.000	*	
0			
0			
9			
0		*	
8	.000	172	
0		*	
0			
7		64	
21	.000	119	
		117	
9		85	
		138	

Total core recovery for interval cored = 77%
Composite RQD = 5%
Average discontinuities/ft = 7.5

C84-04

N 710968.9
E 1743690.5
INCLINATION: VERTICAL

ELEV.	DPTH	LOG	CORE REC.	RQD %	K ft/day	I_s psi	DESCRIPTION
2064.8	0.0						Qal: alluvium; unconsolidated sand, gravel, cobbles, and occasional boulders to 2.5-foot maximum diameter; clayey sand from 54.5-56.5' and 61-62.5'; clayey sand, gravel, and cobbles from 30.5-34'.
Water Level SEPT '84							
2056.5	8.3						
2001.3	63.5						
	65.5						qd: diorite; speckled light greyish-white and dark grey to black; medium grained; highly weathered and soft to moderately hard from 65.5-72'; slightly to moderately weathered and moderately hard to hard from 72-102.4'; highly fractured at random orientations; fracture surfaces smooth to rough; apparent joint sets dip 45-50 and 65-70 degrees; many joints healed with silica and/or calcite, others with evidence of movement; some shears with minor gouge; no samples lab tested.
				0			
				0			
				0		148	
				32	.000	159	
				76		217	
				0	.000	106	
				52		212	
				53		164	
				64	.000	217	
1962.4	102.4					127	

0—100%

Total core recovery for interval cored = 96%
Composite RQD = 38%
Average discontinuities/ft = 6.9

GENERAL NOTES:

1. See plate A-9 for locations and types of exploratory holes.
2. See plate A-30 for an explanation of RQD.
3. See section 5 of this appendix, INVESTIGATION PROCEDURES - Drilling, for the criteria used to describe rock weathering.
4. See section 7 of this appendix for results of laboratory tests.
5. See plates A-30 to A-35 for a graphical representation of rock quality parameters including RQD, discontinuity frequency, point load strength, and degree of weathering.
6. The I_s column shows 50mm size-corrected point load strength data. An asterisk denotes an invalid test.
7. The K column represents rockmass permeability as determined from packer tests.

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

DRILL LOGS
STREAMBOTTOM

SUBMITTED BY: DATE APPROVED:
CHIEF, ENGINEERING DIVISION

SPEC. NO. DACW09-____-____
DISTRICT FILE NO.

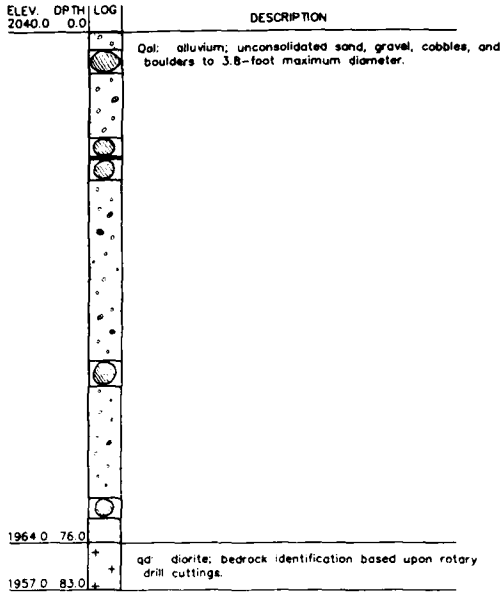
SHEET
2
OF
7
SHEETS

PLATE A-12

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

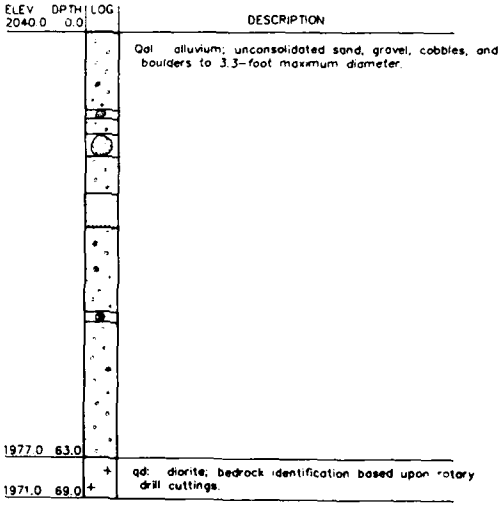
R86-1A

N 710120
E 1743320
INCLINATION: VERTICAL



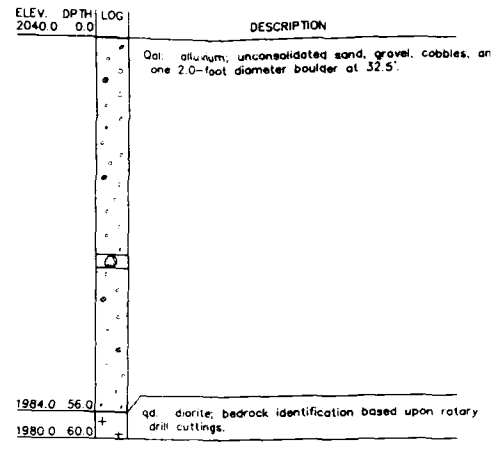
R86-1B

N 710125
E 1743310
INCLINATION: VERTICAL



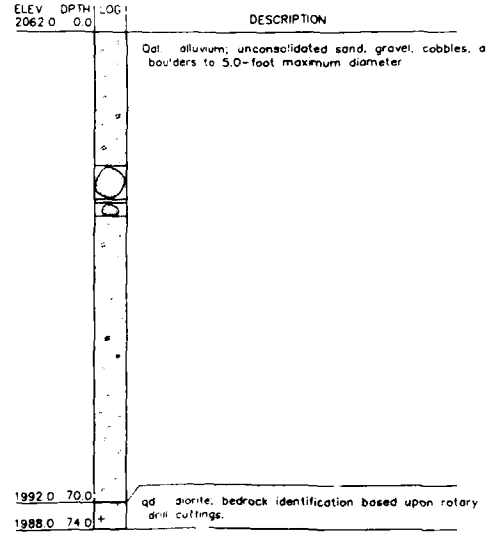
R86-1C

N 710128
E 1743300
INCLINATION: VERTICAL



R86-2A

N 711020
E 1743490
INCLINATION: VERTICAL



LUE ENGINEERING PAYS

86-1C

710128
1743300
LINATION: VERTICAL

DESCRIPTION

Qal: alluvium; unconsolidated sand, gravel, cobbles, and one 2.0-foot diameter boulder at 32.5'.

qd: diorite; bedrock identification based upon rotary drill cuttings.

86-2A

711020
743490
CLINATION: VERTICAL

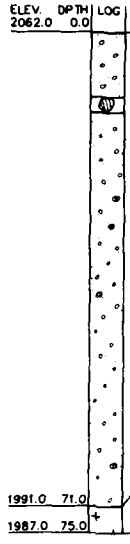
DESCRIPTION

Qal: alluvium; unconsolidated sand, gravel, cobbles, and boulders to 5.0-foot maximum diameter

qd: diorite; bedrock identification based upon rotary drill cuttings.

R86-2B

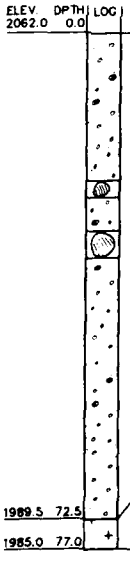
N 711025
E 1743485
INCLINATION: VERTICAL



ELEV.	DPTH	LOG	DESCRIPTION
2062.0	0.0		Qal: alluvium; unconsolidated sand, gravel, cobbles, and one 2.3-foot diameter boulder at 9.7'.
1991.0	71.0		qd: diorite; bedrock identification based upon rotary drill cuttings.
1987.0	75.0		

R86-2C

N 711030
E 1743475
INCLINATION: VERTICAL



ELEV.	DPTH	LOG	DESCRIPTION
2062.0	0.0		Qal: alluvium; unconsolidated sand, gravel, cobbles, and boulders to 4.0-foot maximum diameter.
1989.5	72.5		qd: diorite; bedrock identification based upon rotary drill cuttings.
1985.0	77.0		

NOTES:

1. See plate A-11 for LITHOLOGIC LEGEND relating to drill logs.
2. See plate A-12 for GENERAL NOTES relating to drill logs.

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

DRILL LOGS
STREAMBOTTOM

SUBMITTED BY: DATE APPROVED:
CHIEF, BRANCH

SPEC. NO. DACW09- ____ B- ____
DISTRICT FILE NO

SHEET 3 OF 7 SHEETS

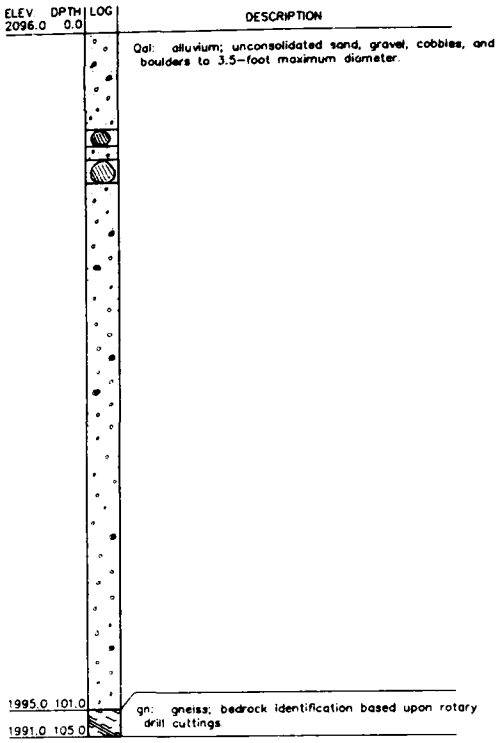
PLATE A-13

SAFETY PAYS

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

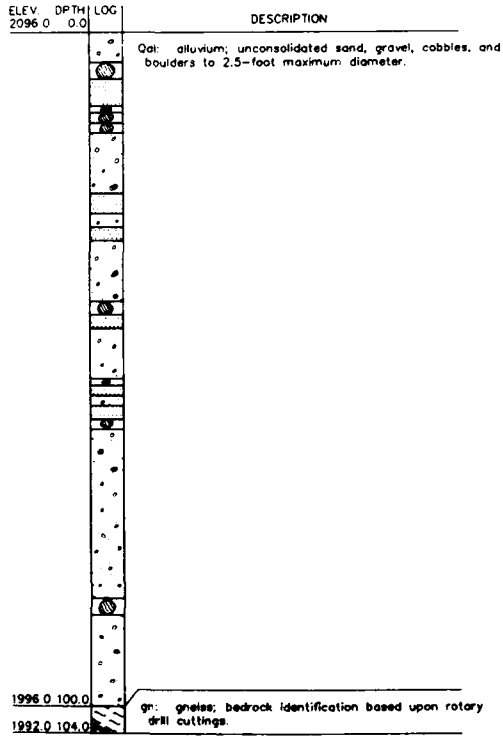
R86-3A

N 711520
E 1744575
INCLINATION: VERTICAL



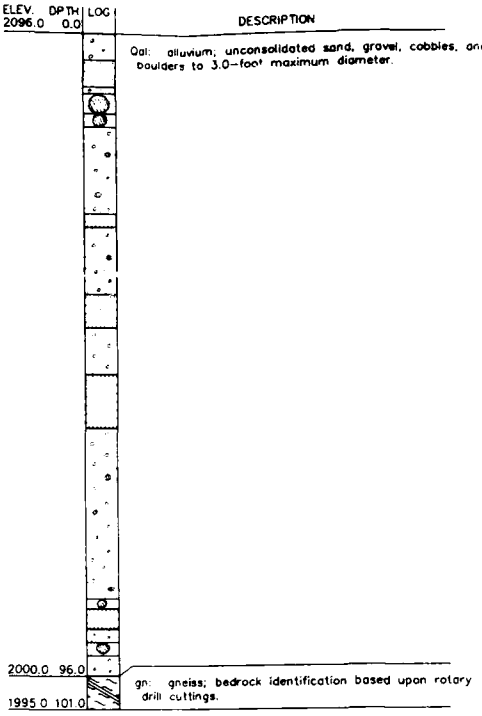
R86-3B

N 711525
E 1744565
INCLINATION: VERTICAL



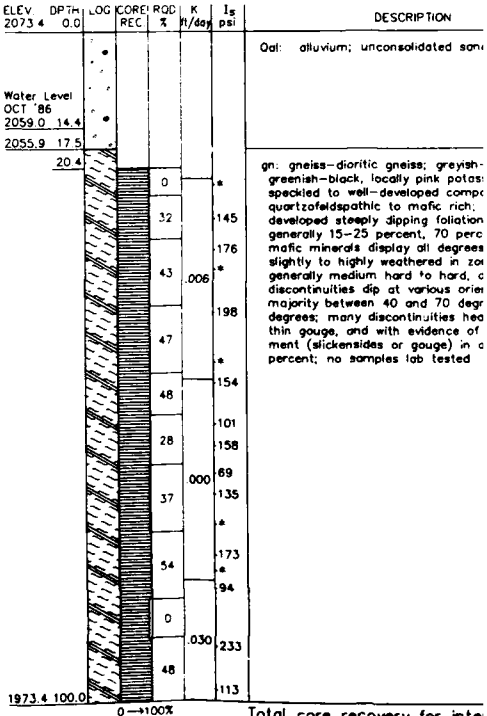
R86-3C

N 711530
E 1744555
INCLINATION: VERTICAL



C86-02

N 710970
E 1744010
INCLINATION: VERTICAL



VALUE ENGINEERING PAYS

R86-3C

N 711530
E 1744555
INCLINATION: VERTICAL

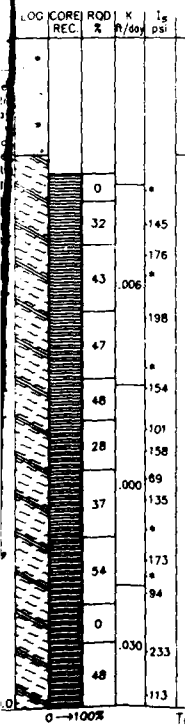
DESCRIPTION

Qal: alluvium; unconsolidated sand, gravel, cobbles, and boulders to 3.0-foot maximum diameter.

gn: gneiss, bedrock identification based upon rotary drill cuttings.

C86-02

N 710970
E 1744010
INCLINATION: VERTICAL



DESCRIPTION

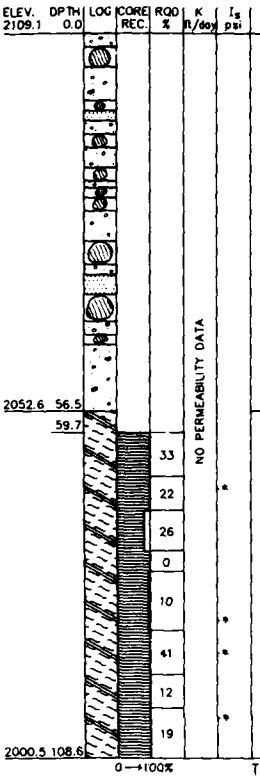
Qal: alluvium; unconsolidated sand, gravel, and cobbles.

gn: gneiss-dioritic gneiss; greyish-white, greyish- to greenish-black, locally pink potassic alteration; speckled to well-developed compositional layering; quartzofeldspathic to mafic rich; poorly to well-developed steeply dipping foliation; mafic composition generally 15-25 percent, 70 percent between 81.7-83.3', mafic minerals display all degrees of chloritization; slightly to highly weathered in zones throughout hole; generally medium hard to hard, occasionally soft; discontinuities dip at various orientations with the majority between 40 and 70 degrees, predominantly 60 degrees; many discontinuities healed or filled with thin gouge, and with evidence of differential movement (slickensides or gouge) in approximately 50 percent; no samples lab tested.

Total core recovery for interval cored = 98%
Composite RQD = 38%
Average discontinuities/ft = 5.5

C86-05

N 711250
E 1745185
INCLINATION: VERTICAL



DESCRIPTION

Qal: alluvium; unconsolidated sand, gravel, cobbles, and occasional boulders to 4-foot maximum diameter.

gn: gneiss; greyish-white, light to dark grey, and black; massive, mottled to well-developed compositional layering generally dipping 70-80 degrees (with one zone dipping 40 degrees); generally fine grained; mafic percentage visually estimated at 5-30 percent, locally up to 50 percent; moderately to highly weathered and soft to very soft; variably-dipping discontinuities with a slight preference for 60 and 90 degrees, 46 percent with slickensides or gouge dipping primarily greater than 40 degrees, apertures generally 0.1-inch-wide or less, wider gouge-filled discontinuities at 99.3-100.5' (2-inch-wide, dipping 80 degrees) and 80.5-81.5' (1-inch-wide, dipping 80 degrees); aplite dikes at 59.7-60.7' (1-inch-wide), 82.6' (0.75-inch-wide), 83.4' (1-inch-wide), and 103' (1-inch-wide); no samples lab tested.

Total core recovery for interval cored = 97%
Composite RQD = 21%
Average discontinuities/ft = 3.6

NOTES:

1. See plate A-11 for LITHOLOGIC LEGEND relating to drill logs.
2. See plate A-12 for GENERAL NOTES relating to drill logs.

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

DRILL LOGS
STREAMBOTTOM

SUBMITTED BY: DATE APPROVED:
CHIEF, BRANCH

SPEC. NO. DACW09- - -
DISTRICT FILE NO.

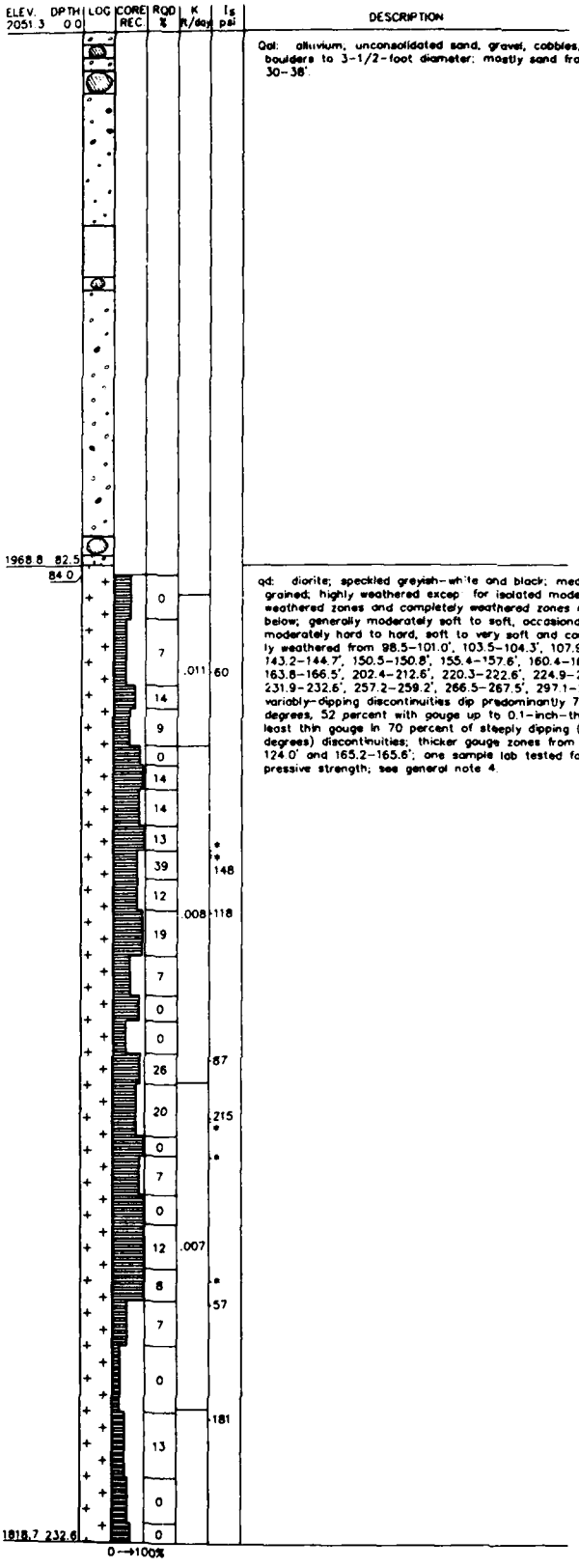
SHEET 4 OF 7 SHEETS

PLATE A-14

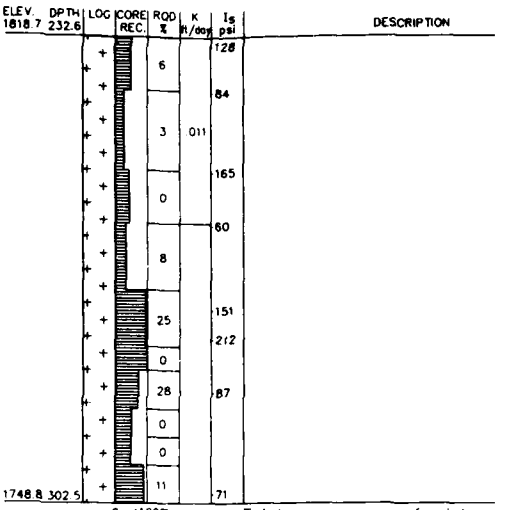
SAFETY PAYS

C86-26

N 710530
E 1743440
INCLINATION: VERTICAL



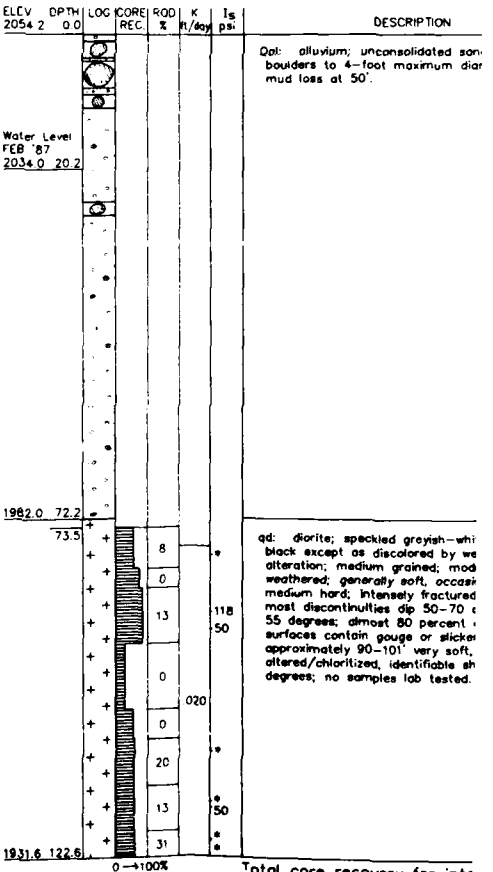
C86-26 (Continued)



Total core recovery for interv
Composite RQD = 9%
Average discontinuities/ft = 6

C86-28

N 710635
E 1743295
INCLINATION: VERTICAL

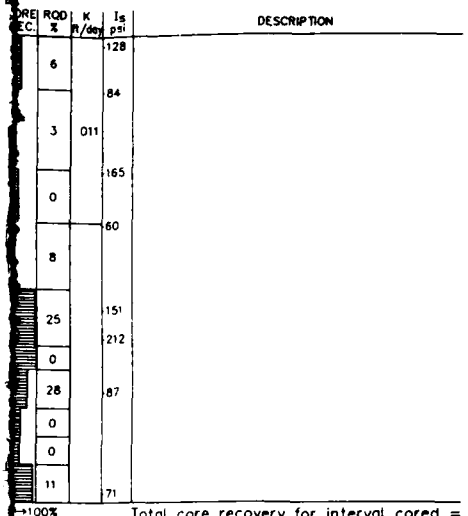


Total core recovery for inte
Composite RQD = 10%
Average discontinuities/ft =

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

VALUE ENGINEERING PAYS

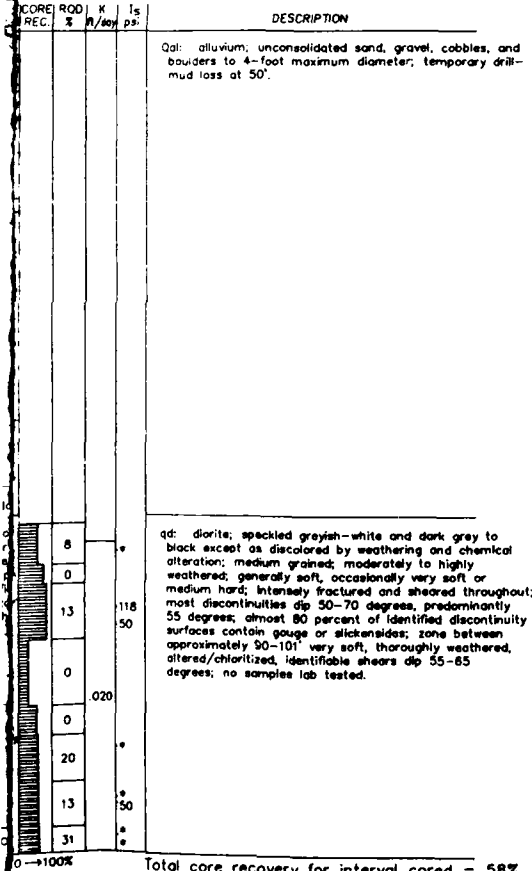
C86-26 (Continued)



Total core recovery for interval cored = 63%
Composite RQD = 9%
Average discontinuities/ft = 6.9

C86-28

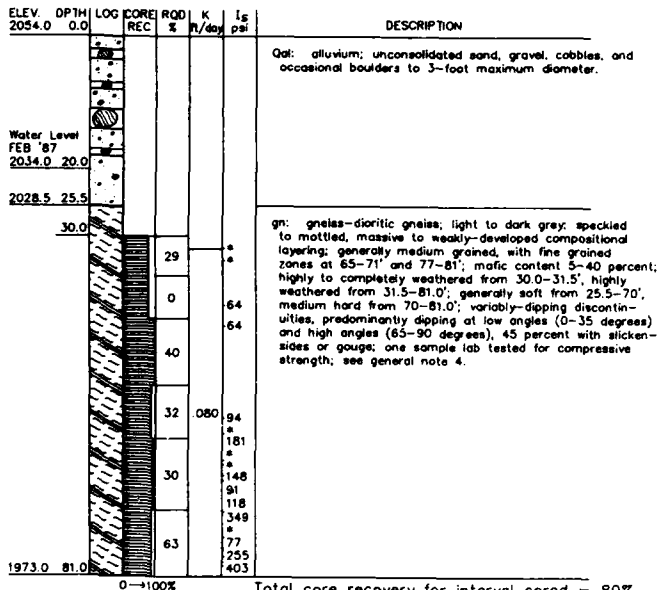
N 710635
E 1743295
INCLINATION: VERTICAL



Total core recovery for interval cored = 58%
Composite RQD = 10%
Average discontinuities/ft = 9.6

C86-29

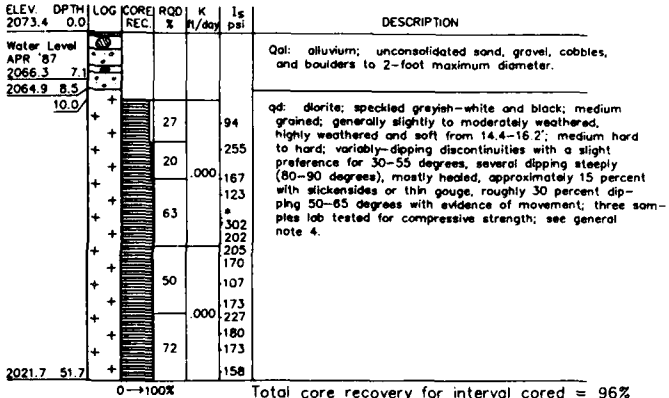
N 710625
E 1743555
INCLINATION: VERTICAL



Total core recovery for interval cored = 89%
Composite RQD = 35%
Average discontinuities/ft = 6.3

C86-32

N 710792
E 1744097
INCLINATION: VERTICAL



Total core recovery for interval cored = 96%
Composite RQD = 51%
Average discontinuities/ft = 3.8

NOTES:

1. See plate A-11 for LITHOLOGIC LEGEND relating to drill logs.
2. See plate A-12 for GENERAL NOTES relating to drill logs.

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

DRILL LOGS
STREAMBOTTOM

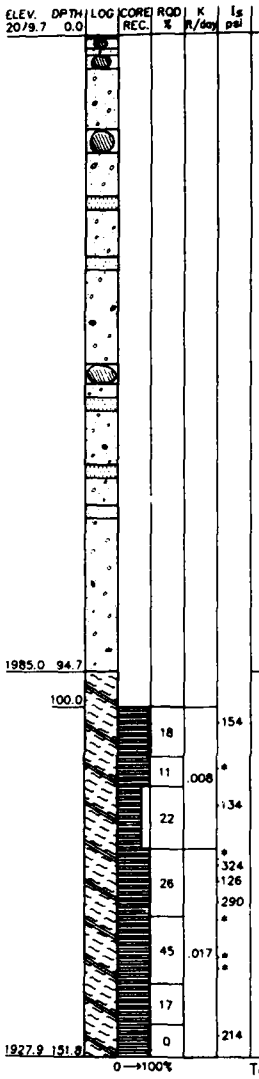
SUBMITTED BY:
DATE APPROVED:
SPEC. NO. DACW09- ---- B- ----
DISTRICT FILE NO.
SHEET 5 OF 7 SHEETS

PLATE A-16

SAFETY PAYS

C86-33

N 711080
E 1744205
INCLINATION: VERTICAL



DESCRIPTION

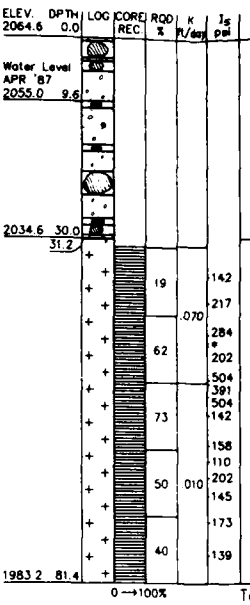
Qal: alluvium; unconsolidated sand, gravel, cobbles, and occasional boulders to 3-foot maximum diameter.

gn: gneiss; greyish-white, grey, pink, and black; speckled, massive to well-developed compositional layering dipping 70-90 degrees; fine and medium grained crystals, with orthoclase feldspar phenocrysts from 100.0-121.7'; estimated 5-20 percent mafic content, mafic-rich (greater than 40 percent) zone from 147.0-147.5'; moderately weathered and soft to medium hard; discontinuities dip predominantly 20-70 degrees, 36 percent with slickensides or gouge (dipping primarily 45-90 degrees); gouge zones at 105.5-107.5' and 145.3-147.0'; quartzofeldspathic dikes at 112.3-112.7' and 113.5-114.6'; no samples lab tested.

Total core recovery for interval cored = 94%
Composite RQD = 23%
Average discontinuities/ft = 3.8

C86-34

N 710915
E 1743800
INCLINATION: VERTICAL



DESCRIPTION

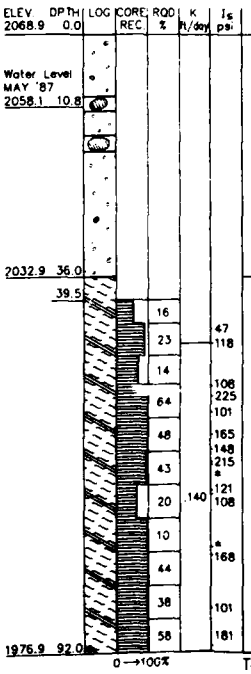
Qal: alluvium; unconsolidated sand, gra
occasional boulders to 3.5-foot maximu

qd: diorite; medium to dark grey; mass
and equigranular; mafic-rich (30-40 p
31.2-41', 51-52.4', and 80-81.4'; sligh
generally medium hard 30-80', highly w
80-81.4'; variably-dipping discontinuitie
dipping at low angles (0-35 degrees) a
(65-90 degrees), overall 12 percent wit
gouge, only 2 percent for discontinuitie
angles; two samples lab tested for com
see general note 4.

Total core recovery for interval
Composite RQD = 49%
Average discontinuities/ft = 3.4

C86-35

N 711205
E 1743760
INCLINATION: VERTICAL



DESCRIPTION

Qal: alluvium; unconsolidated sand, gra
boulders to 2.4-foot maximum diamet

gn: gneiss; greyish-white to greyish-
speckled to slightly banded; medium g
to completely weathered and soft to v
52.1'); slightly to highly weathered and
soft (52.1-92.0'); estimated 20-40 p
variably-dipping discontinuities with a
ence for 35 degrees, about 50 percen
filling; no samples lab tested.

Total core recovery for interval
Composite RQD = 35%
Average discontinuities/ft = 6.6

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

VALUE ENGINEERING PAYS

C86-34

N 710915
E 1743800
INCLINATION: VERTICAL

CORE REC. %	RQD %	K ft/day	I_s psi	DESCRIPTION
				Qal: alluvium; unconsolidated sand, gravel, cobbles, and occasional boulders to 3.5-foot maximum diameter.
19			142	qd: diorite; medium to dark grey, massive; medium grained and equigranular; mafic-rich (30-40 percent) zones at 31.2-41', 51-52.4', and 80-81.4'; slightly weathered and generally medium hard 30-80', highly weathered and soft 80-81.4'; variably-dipping discontinuities, predominantly dipping at low angles (0-35 degrees) and at high angles (65-90 degrees), overall 12 percent with slickensides or gouge, only 2 percent for discontinuities dipping at low angles; two samples lab tested for compressive strength; see general note 4.
		.070	217	
62			284	
			* 202	
			504	
			391	
73			504	
			142	
			158	
			110	
50		.010	202	
			145	
			173	
40			139	
→100%				Total core recovery for interval cored = 100% Composite RQD = 49% Average discontinuities/ft = 3.4

C86-35

N 711205
E 1743760
INCLINATION: VERTICAL

CORE REC. %	RQD %	K ft/day	I_s psi	DESCRIPTION
				Qal: alluvium; unconsolidated sand, gravel, cobbles, and boulders to 2.4-foot maximum diameter.
16			47	gn: gneiss; greyish-white to greyish- or greenish-black; speckled to slightly banded; medium grained; moderately to completely weathered and soft to very soft (39.5-52.1'); slightly to highly weathered and medium hard to soft (52.1-92.0'); estimated 20-40 percent mafics; variably-dipping discontinuities with a slight preference for 35 degrees, about 50 percent with gouge or clay filling; no samples lab tested.
23			118	
14			108	
64			225	
			101	
48			165	
			148	
43			215	
			* 121	
20		.140	108	
10			* 168	
44				
38			101	
58			181	
→100%				Total core recovery for interval cored = 89% Composite RQD = 35% Average discontinuities/ft = 6.6

C86-36

N 711050
E 1744015
INCLINATION: VERTICAL

ELEV.	DPTH	LOG	CORE REC. %	RQD %	K ft/day	I_s psi	DESCRIPTION
Water Level APR '87							
2065.0	5.7						Qal: alluvium; unconsolidated sand, gravel, cobbles, and occasional boulders to 2-foot maximum diameter.
2017.0	53.7						
	59.5			14		121	gn: gneiss; greyish-white, grey, black, and pink; speckled, mottled, and poorly- to well-foliated with foliations dipping 45-75 degrees; estimated 5-40 percent mafic content, mafic-rich zones at 67-72' (mafic content 30-50 percent) and 108-108.5' (mafic content greater than 50 percent); generally highly weathered except completely weathered from 108-108.5'; generally soft; discontinuities dip predominantly 30-90 degrees, 63 percent with slickensides or gouge; significant chloritic alteration at 85.5'; no samples lab tested.
				28	.000	* 128	
						101	
				16			
				40		124	
						97	
				0		114	
				0			
				0	.000	104	
				0			
				14		131	
				35	.014	* 232	
1961.2	109.5					215	
			0→100%			114	Total core recovery for interval cored = 87% Composite RQD = 18% Average discontinuities/ft = 6.5

NOTES:

1. See plate A-11 for LITHOLOGIC LEGEND relating to drill logs.
2. See plate A-12 for GENERAL NOTES relating to drill logs.

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

DRILL LOGS
STREAMBOTTOM

SUBMITTED BY: DATE APPROVED:

SPEC. NO. DACW09-____-____

DISTRICT FILE NO.

SHEET 6 OF 7 SHEETS

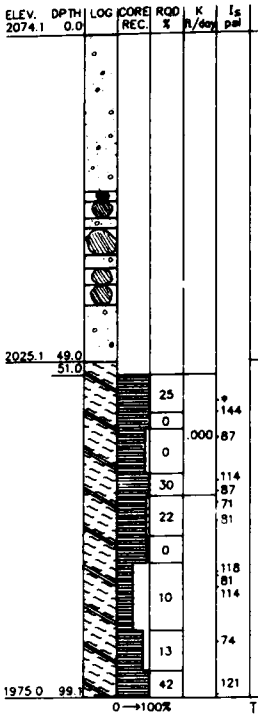
PLATE A-10

SAFETY PAYS

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

C86-37

N 711210
E 1744010
INCLINATION: VERTICAL



DESCRIPTION

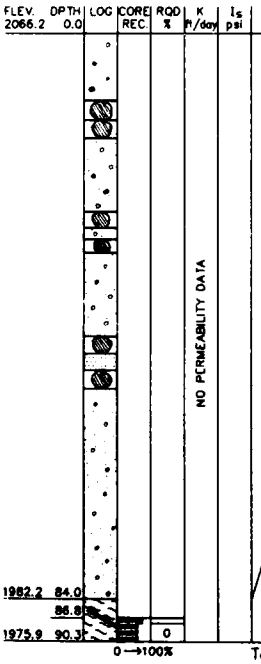
Qal: alluvium; unconsolidated sand, gravel, cobbles, and occasional boulders to 4-foot maximum diameter.

gn: gneiss; grayish-white, gray to dark grey, and pink and black; speckled, mottled, and poorly- to well-foliated with foliations dipping 70-90 degrees; generally medium grained, aphanitic zones at 56.7-58.5' and 75-77', porphyroblastic from 95-97'; estimated 2-40 percent mafic content, mafic-rich zones at 58.5-59.1' (greater than 40 percent), 65.8-66.5' (30 percent), 75-77' (greater than 40 percent), and 83-89' (greater than 60 percent); highly to completely weathered; soft to very soft; variably-dipping discontinuities, overall 36 percent with slickensides or gouge, 68 percent steeper than 45 degrees with slickensides; no samples lab tested.

Total core recovery for interval cored = 83%
Composite RQD = 15%
Average discontinuities/ft = 5.5

C86-38

N 711160
E 1743760
INCLINATION: VERTICAL



DESCRIPTION

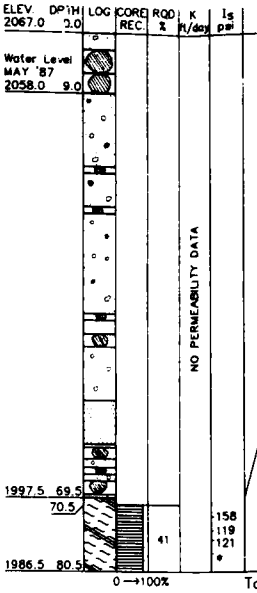
Qal: alluvium; unconsolidated sand, gravel, cobbles, and boulders to 2.9-foot maximum diameter.

gn: gneiss; greyish- to pinkish-white to greenish-grey; massive; medium grained; highly to completely weathered and soft to very soft; estimated 10-15 percent mafics (86.8-88.8'), 90 percent chloritized mafics (88.8-89.2'); no samples lab tested.

Total core recovery for interval cored = 0%
Composite RQD = 0%
Average discontinuities/ft = 0.0

C86-39

N 711040
E 1743890
INCLINATION: VERTICAL



DESCRIPTION

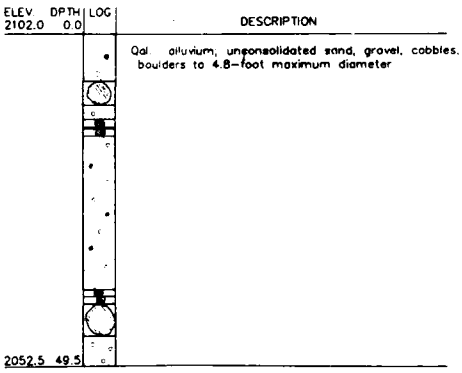
Qal: alluvium, unconsolidated sand, gra
occasional boulders to 3.5-foot maxim
mud loss between 11-16'.

gn: gneiss; light brown; generally mas
very slight compositional layering; fine
estimated 5-20 percent mafic content
(greater than 40 percent) zone at 78-
weathered; soft to moderately hard, ve
80.5'; discontinuities dip predominantly
grees, approximately 30 percent with s
or gouge, no samples lab tested

Total core recovery for interval
Composite RQD = 41%
Average discontinuities/ft = 4.6

R86-40

N 711625
E 1744770
INCLINATION: VERTICAL

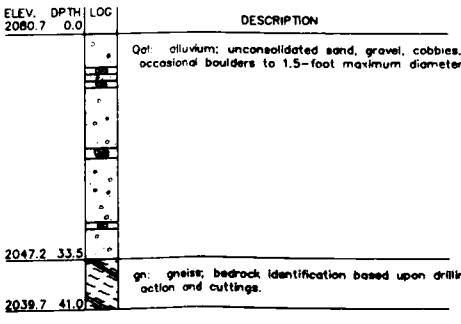


DESCRIPTION

Qal: alluvium; unconsolidated sand, gravel, cobbles, and boulders to 4.8-foot maximum diameter

R86-41

N 711100
E 1744310
INCLINATION: VERTICAL



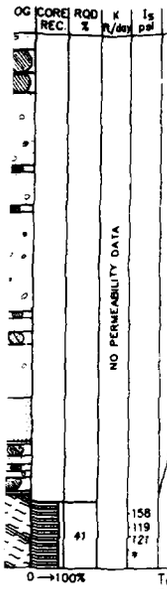
DESCRIPTION

Qal: alluvium; unconsolidated sand, gravel, cobbles, and occasional boulders to 1.5-foot maximum diameter

gn: gneiss; bedrock identification based upon drilling action and cuttings.

C86-39

N 711040
E 1743890
INCLINATION: VERTICAL



DESCRIPTION

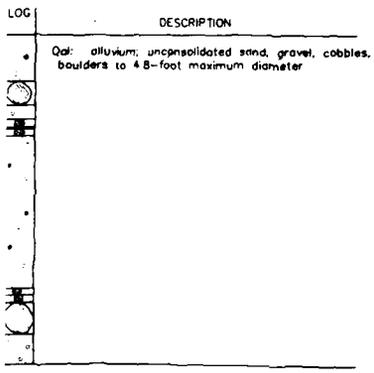
Qal: alluvium; unconsolidated sand, gravel, cobbles, and occasional boulders to 3.5-foot maximum diameter; mud loss between 11-16'.

gn: gneiss; light brown; generally massive, speckled; very slight compositional layering; fine grained; estimated 5-20 percent mafic content, mafic-rich (greater than 40 percent) zone at 78-80.5'; highly weathered; soft to moderately hard, very soft 78-80.5'; discontinuities dip predominantly 25-65 degrees, approximately 30 percent with slickensides or gouge; no samples lab tested.

Total core recovery for interval cored = 83%
Composite RQD = 41%
Average discontinuities/ft = 4.6

R86-40

N 711625
E 1744770
INCLINATION: VERTICAL

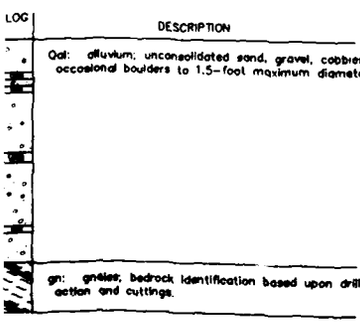


DESCRIPTION

Qal: alluvium; unconsolidated sand, gravel, cobbles, and boulders to 4.8-foot maximum diameter

R86-41

N 711100
E 1744310
INCLINATION: VERTICAL



DESCRIPTION

Qal: alluvium; unconsolidated sand, gravel, cobbles, and occasional boulders to 1.5-foot maximum diameter.

gn: gneiss; bedrock identification based upon drilling action and cuttings.

NOTES:

1. See plate A-11 for LITHOLOGIC LEGEND relating to drill logs.
2. See plate A-12 for GENERAL NOTES relating to drill logs.

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY
DRAWN BY
CHECKED BY

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

DRILL LOGS
STREAMBOTTOM

SUBMITTED BY: DATE APPROVED:
CHIEF, BRANCH

SPEC. NO. DACW09-___-B-___
DISTRICT FILE NO.

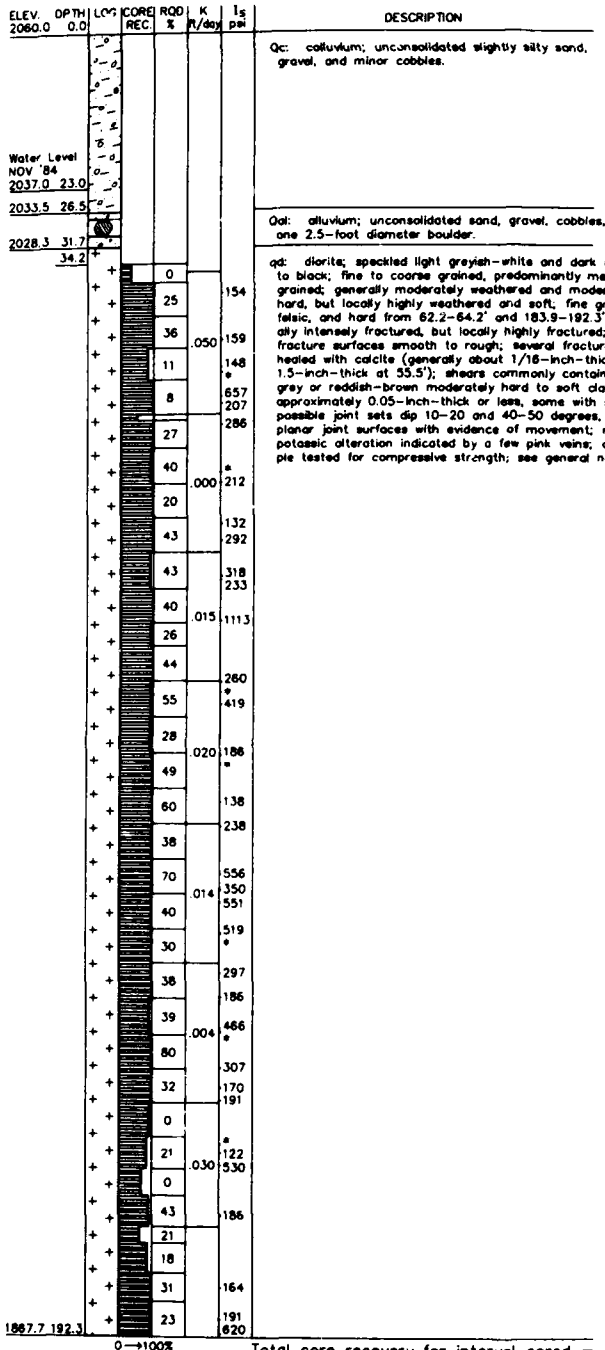
SHEET 7 OF 7 SHEETS

PLATE A-17

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

C84-06

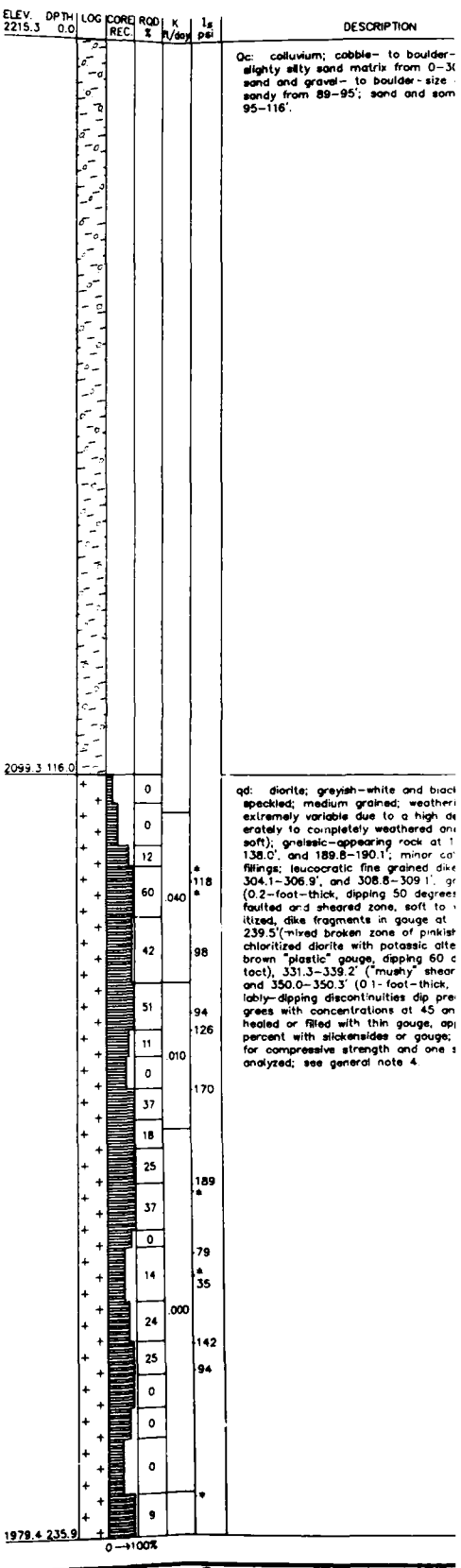
N 710509.5
E 1743830.2
INCLINATION: VERTICAL



Total core recovery for interval cored = 93%
Composite RQD = 34%
Average discontinuities/ft = 5.8

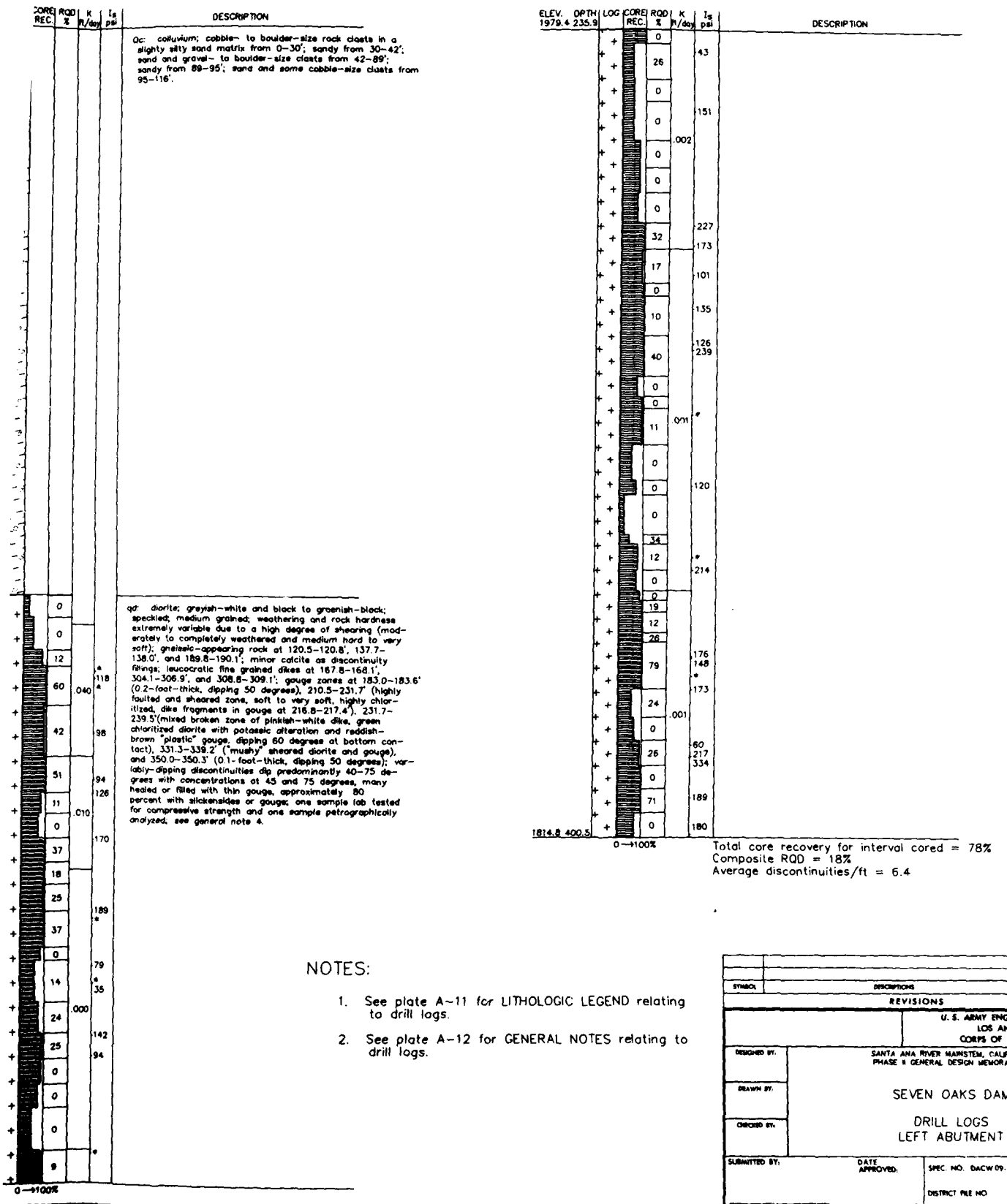
C86-04

N 710710
E 1744365
INCLINATION: VERTICAL



C86-04
N 710710
E 1744365
INCLINATION: VERTICAL

C86-04 (Continued)



Total core recovery for interval cored = 78%
Composite RQD = 18%
Average discontinuities/ft = 6.4

NOTES:

1. See plate A-11 for LITHOLOGIC LEGEND relating to drill logs.
2. See plate A-12 for GENERAL NOTES relating to drill logs.

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

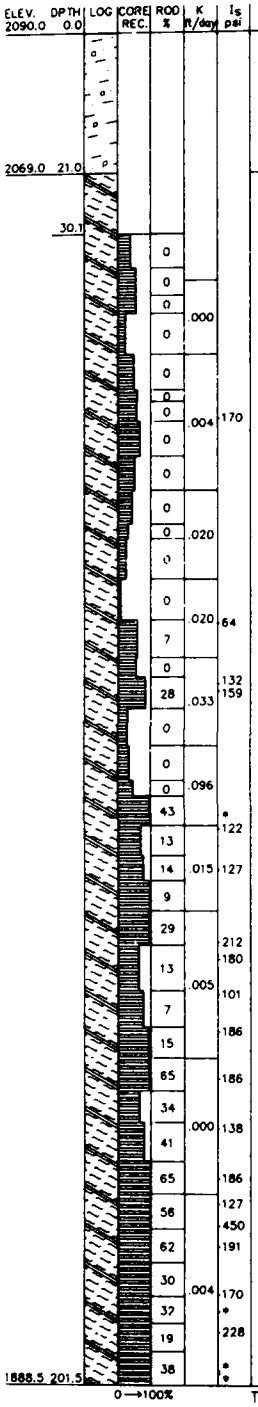
DRILL LOGS
LEFT ABUTMENT

SUBMITTED BY: DATE APPROVED:
SPEC. NO. DACW09-____-B-____
DISTRICT FILE NO.
SHEET

PLATE A-18

C84-07

N 711250
E 1743270
INCLINATION: VERTICAL



DESCRIPTION

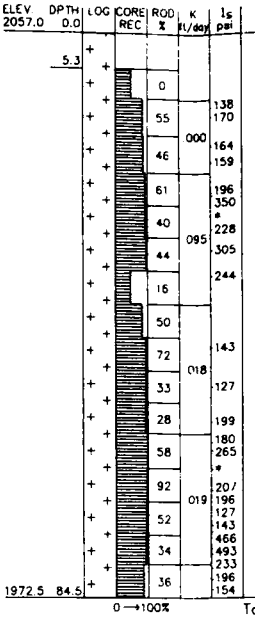
Qc: colluvium; unconsolidated slightly silty sand with minor gravel.

gn: gneiss; greyish-white and greyish- to greenish-black with pink potassic alteration; fine to medium grained; generally completely weathered and soft from 30.1-53', generally moderately weathered and moderately hard but locally completely weathered and soft from 53-153, generally unweathered to slightly weathered from 153-201.5'; steeply-dipping foliation, more distinct from 176-201.5'; quartzofeldspathic to mafic- rich, mafics commonly altered to clay from 30.1-53'; intensely fractured and sheared from 30.1-153', highly fractured from 153-201.5', with fractures dipping at random orientations; fracture surfaces smooth to rough, commonly containing a clay coating; possible joint sets dip 10-20, 40-45, and 60-65 degrees, most planar joint surfaces with evidence of movement; no samples lab tested

Total core recovery for interval cored = 67%
Composite RQD = 18%
Average discontinuities/ft = 7.6

C84-08

N 710860
E 1743160
INCLINATION: 47° (from horizonta
BEARING: N45W



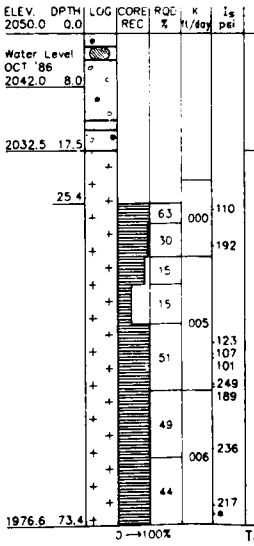
DESCRIPTION

qd: diorite; speckled light greyish-white to black; medium grained; moderately ately hard, but locally highly weathered, and soft from 37-42'; highly to inters fracture surfaces smooth to rough; int fault breccia and gouge from 41-50'; dip 45-50 degrees, many planar joint with slickensides; minor calcite at 53.1' samples tested for compressive strengt

Total core recovery for interval
Composite RQD = 45%
Average discontinuities/ft = 5.4

C86-01

N 710800
E 1743175
INCLINATION: VERTICAL



DESCRIPTION

Qal: alluvium; unconsolidated sand, gra boulders to 2-foot maximum diameter

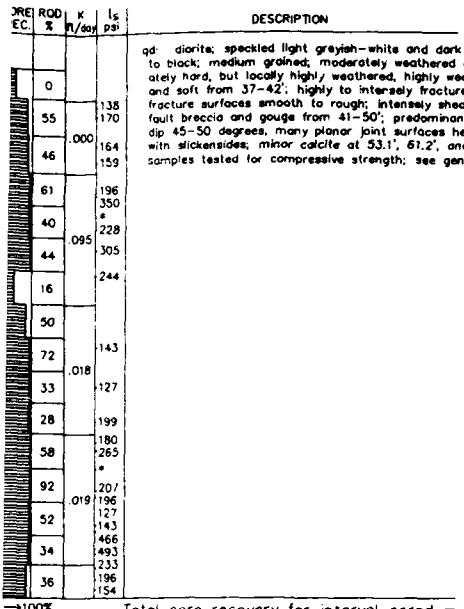
qd: diorite; speckled greyish-white and grained; slightly weathered; recovered hard to hard, softer 59-61'; discontin angles, evidence of movement in rough most steeply-dipping discontinuities (7 contain gouge; gouge zones to 0.5 in and 73' dip 80 and 75 degrees, respe thick, near-horizontal green gouge at tested for compressive strength; see

Total core recovery for interval
Composite RQD = 40%
Average discontinuities/ft = 4.4

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

C84-08

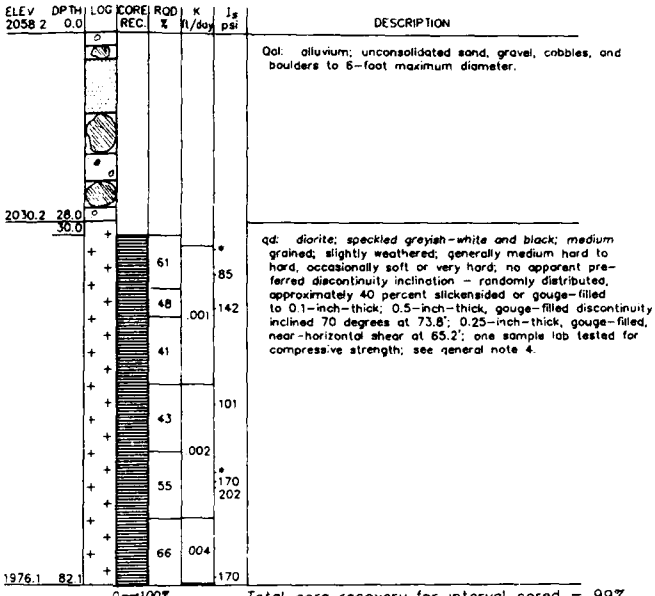
N 710860
E 1743160
INCLINATION: 47° (from horizontal)
BEARING: N45W



Total core recovery for interval cored = 88%
Composite RQD = 45%
Average discontinuities/ft = 5.4

C86-03

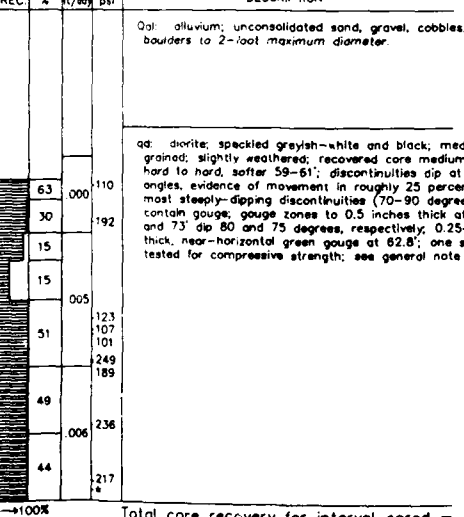
N 710955
E 1743260
INCLINATION: VERTICAL



Total core recovery for interval cored = 99%
Composite RQD = 52%
Average discontinuities/ft = 3.6

C86-01

N 710800
E 1743175
INCLINATION: VERTICAL



Total core recovery for interval cored = 90%
Composite RQD = 40%
Average discontinuities/ft = 4.4

NOTES:

1. See plate A-11 for LITHOLOGIC LEGEND relating to drill logs.
2. See plate A-12 for GENERAL NOTES relating to drill logs.

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

DRILL LOGS
RIGHT ABUTMENT

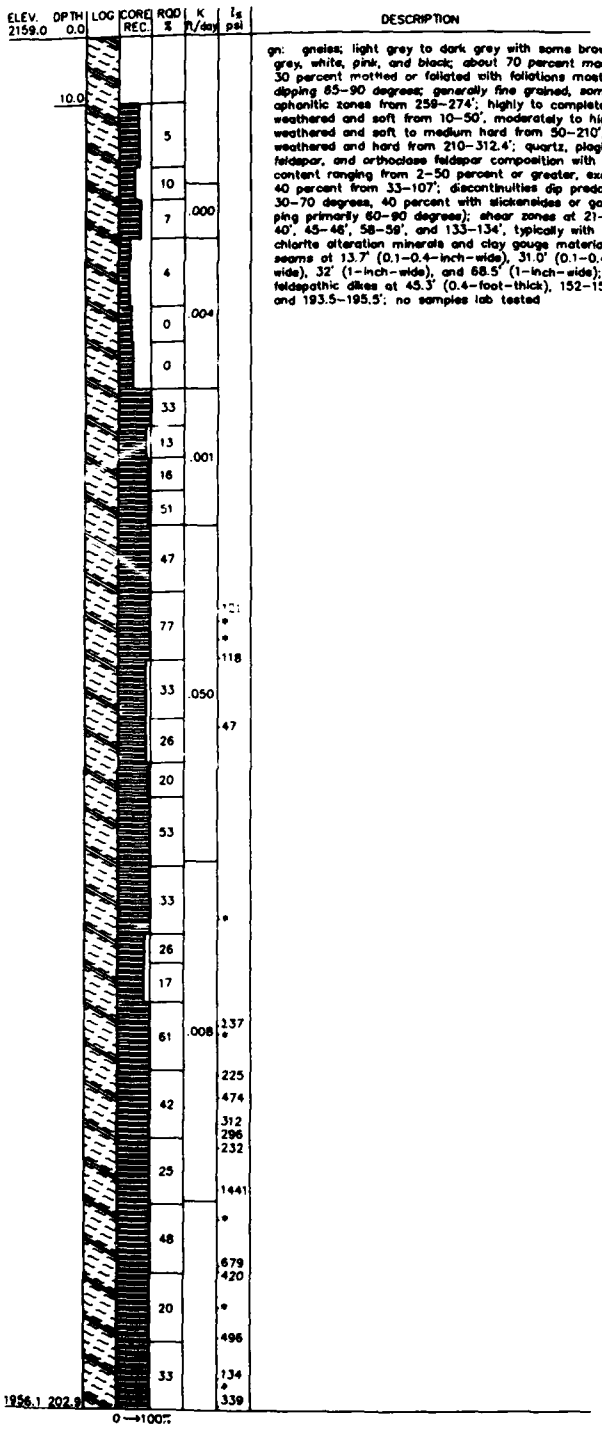
SUBMITTED BY: DATE APPROVED:
CHIEF, ENGINEERING DIVISION

SPEC. NO. DACW09-___-B-___
DISTRICT FILE NO.

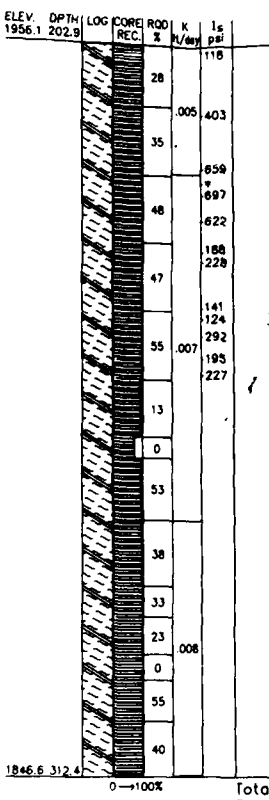
SHEET 1 OF 2 SHEETS

PLATE A-10

C86-23
N 711260
E 1743115
INCLINATION: VERTICAL



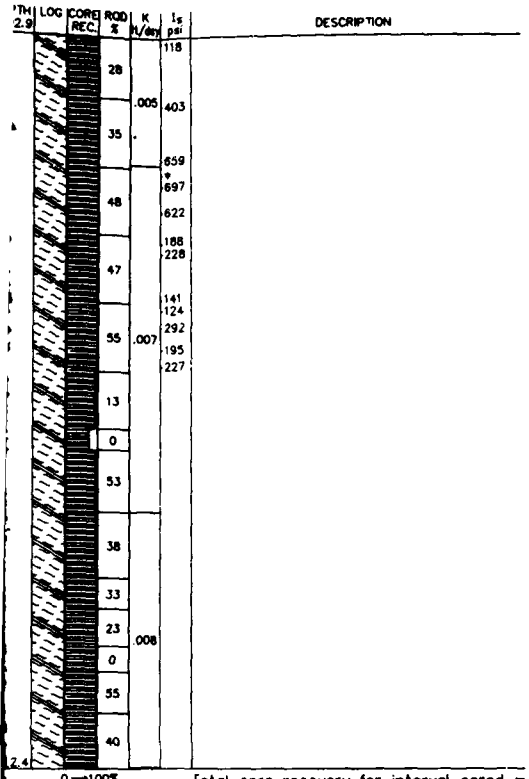
C86-23 (Continued)



Total core recovery for interval co
Composite RQD = 33%
Average discontinuities/ft = 3.4

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

C86-23 (Continued)



total core recovery for interval cored = 90%
Composite RQD = 33%
Average discontinuities/ft = 3.4

NOTES:

1. See plate A-11 for LITHOLOGIC LEGEND relating to drill logs.
2. See plate A-12 for GENERAL NOTES relating to drill logs.

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

DRILL LOGS
RIGHT ABUTMENT

SUBMITTED BY:
DATE APPROVED:
CHIEF, ENGINEERING DIVISION

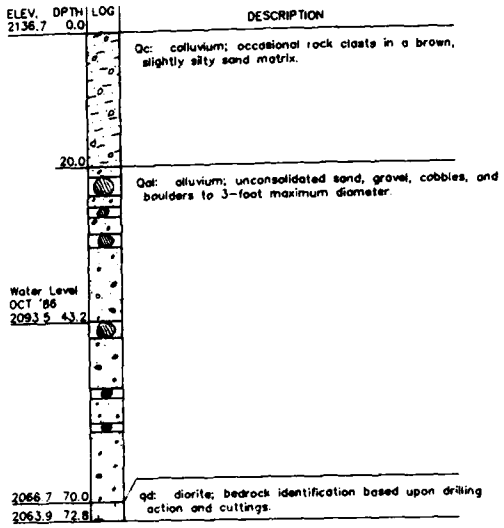
SPEC. NO. DACW09-___-B-____
DISTRICT FILE NO.

SHEET 2 OF 2 SHEETS

PLATE A-20

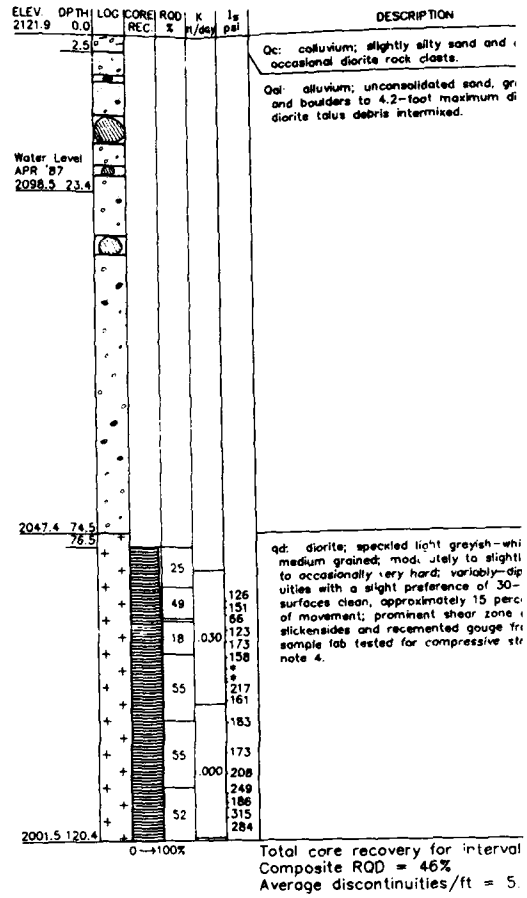
R86-06

N 710905
E 1745368
INCLINATION: VERTICAL



C86-30

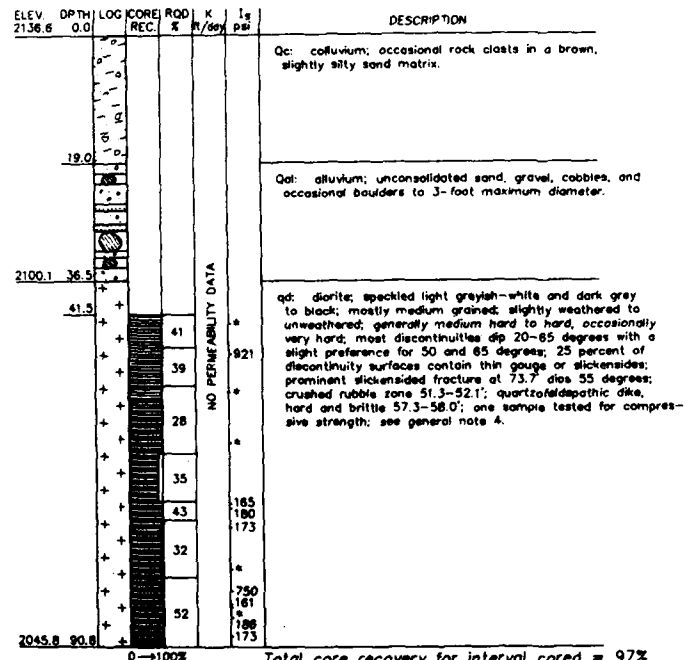
N 710957
E 1745372
INCLINATION: VERTICAL



Total core recovery for interval
Composite RQD = 46%
Average discontinuities/ft = 5.

C86-07

N 710881
E 1745340
INCLINATION: VERTICAL



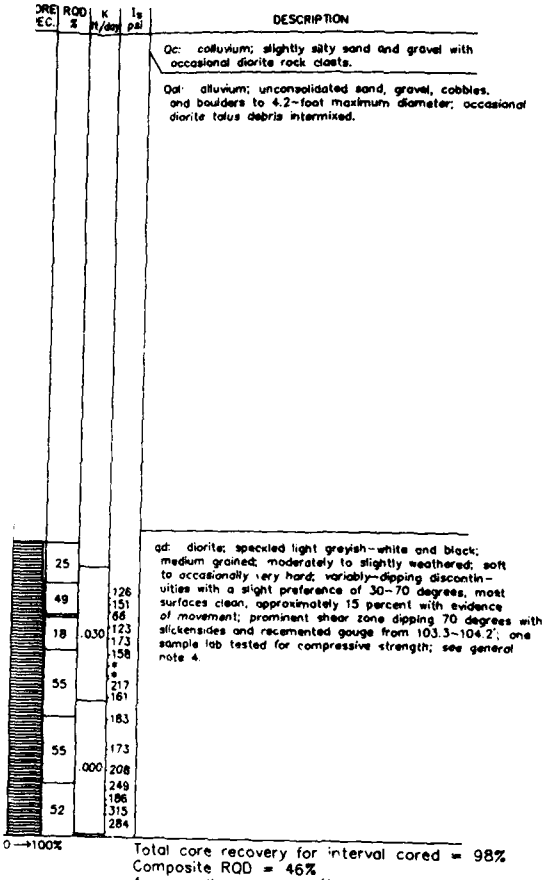
Total core recovery for interval cored = 97%
Composite RQD = 38%
Average discontinuities/ft = 3.1

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

ALUE ENGINEERING PAYS

C86-30

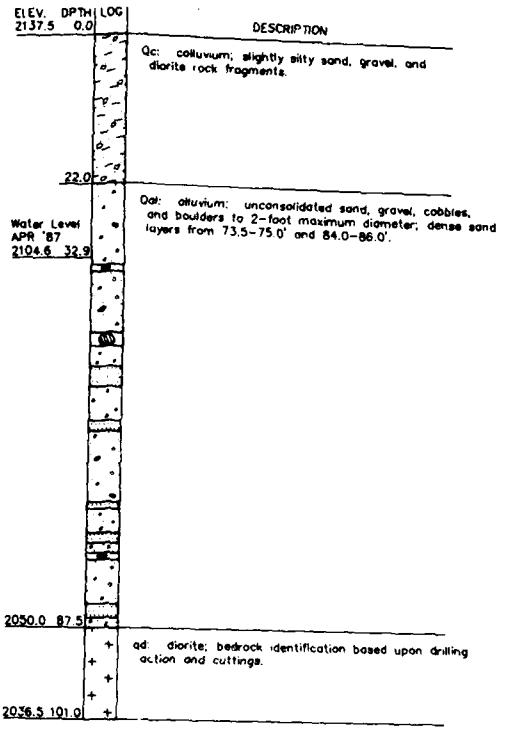
N 710957
E 1745372
INCLINATION: VERTICAL



Total core recovery for interval cored = 98%
Composite RQD = 46%
Average discontinuities/ft = 5.4

R86-31

N 710906
E 1745418
INCLINATION: VERTICAL



NOTES:

1. See plate A-11 for LITHOLOGIC LEGEND relating to drill logs.
2. See plate A-12 for GENERAL NOTES relating to drill logs.

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM
DRILL LOGS
OUTLET WORKS
INLET PORTAL

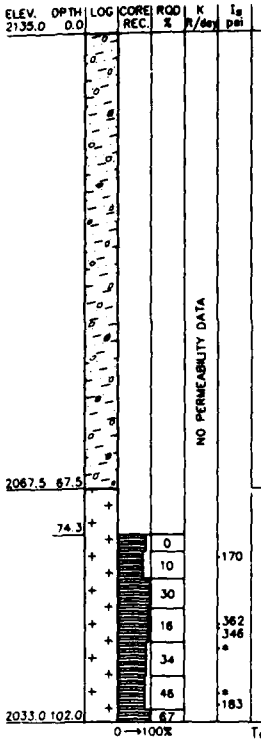
SUBMITTED BY:
DATE APPROVED:
SPEC. NO. DACW09-____-____
DISTRICT FILE NO.
SHEET
DWG
SHEET

PLATE A-21

SAFETY PAYS

C86-12

N 709665
E 1744005
INCLINATION: VERTICAL



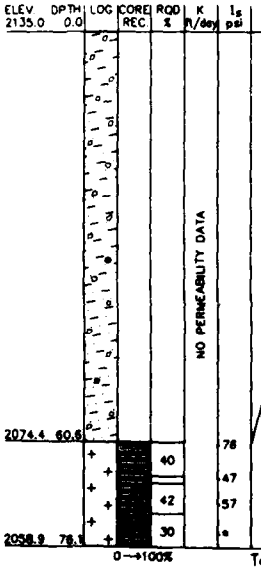
Qc: colluvium; abundant cobble- to boulder-size dioritic rock clasts in a slightly silty sand matrix.

qd: diorite; speckled greyish-white and black; medium grained; increased mafic content 90.2-96.0'; moderately weathered to 76' then variably unweathered to slightly weathered; generally moderately hard to occasionally hard; preferred dip of discontinuities 30 and 60 degrees, apertures generally less than 0.1-inch wide and completely filled; evidence of differential movement in roughly 30 percent of discontinuities; no samples lab tested.

Total core recovery for interval cored = 88%
Composite RQD = 28%
Average discontinuities/ft = 3.9

C86-13

N 709648
E 1744020
INCLINATION: VERTICAL



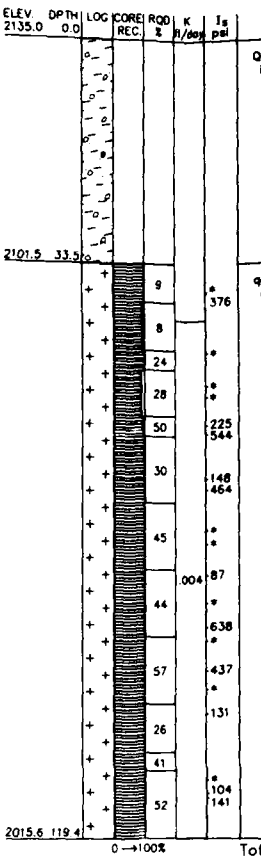
Qc: colluvium; abundant diorite rock clasts to 10-inch diameter in a slightly silty sand matrix.

qd: diorite; speckled greyish-white to brownish grey-white and black; medium grained; slightly to moderately weathered; soft to medium hard; majority of discontinuities dip steeper than 45 degrees with a preferred dip of 60 degrees, apertures generally less than 0.1-inch-wide; evidence of differential movement in approximately 50 percent of discontinuities; rubble crushed zone with minor gouge 63.9-65.2'; moderately weathered shear zone with slickensides and gouge 70.2-71.3'; no samples lab tested.

Total core recovery for interval cored = 97%
Composite RQD = 38%
Average discontinuities/ft = 3.3

C86-24

N 709678
E 1743994
INCLINATION: 60° (from horizon
BEARING: N50E



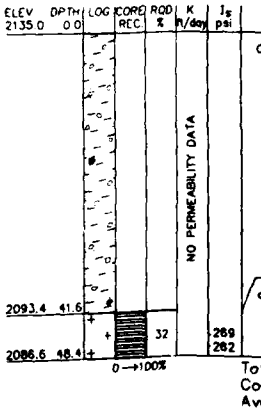
Qc: colluvium; diorite rock clasts to
in a brown, slightly silty sand matrix;

qd: diorite; speckled greyish-white a
grained; minor secondary calcite 79-
weathered to 34.5' then slightly weat
noted below; generally moderately ha
discontinuities inclined at all angles
preference of 10, 30, 40, and 45 de
axis; surfaces generally clean; less t
slickensided; highly to moderately we
103.3-106.6', slight chloritization, 20
prominent surfaces inclined 60-70 d
axis; rock rubble 39.3-41.0' and 50.
lab tested.

Total core recovery for interva
Composite RQD = 36%
Average discontinuities/ft = 5

C86-25

N 709675
E 1743991
INCLINATION: VERTICAL



Qc: colluvium; cobble-size diorite r
slightly silty sand matrix.

qd: diorite; speckled greyish-white
grained; highly to moderately weat
hard; variably-dipping discontinuiti
rough and clean (but varying to s
no samples lab tested.

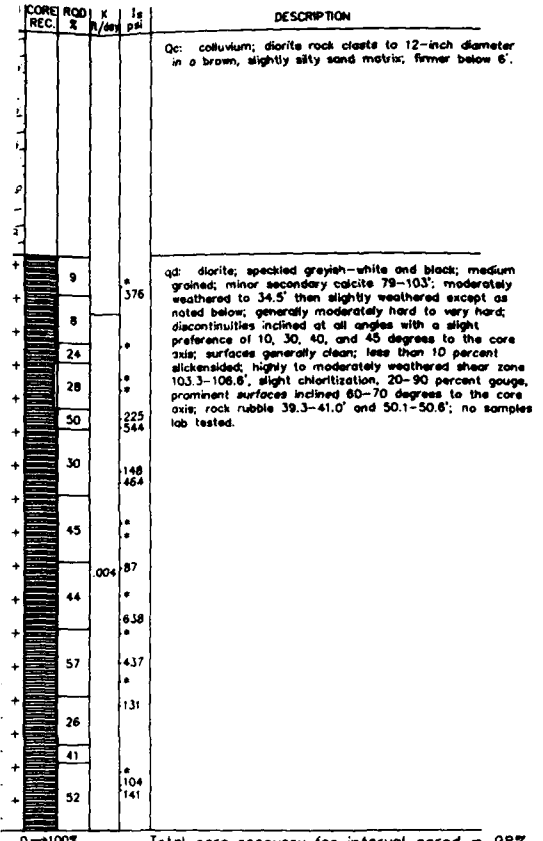
Total core recovery for inter
Composite RQD = 32%
Average discontinuities/ft =

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

VALUE ENGINEERING PAYS

C86-24

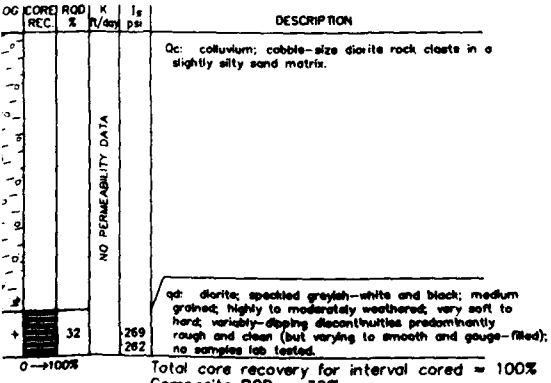
N 709678
E 1743994
INCLINATION: 60° (from horizontal)
BEARING: N50E



Total core recovery for interval cored = 98%
Composite RQD = 36%
Average discontinuities/ft = 5.1

C86-25

N 709675
E 1743991
INCLINATION: VERTICAL



Total core recovery for interval cored = 100%
Composite RQD = 32%
Average discontinuities/ft = 4.4

NOTES:

1. See plate A-11 for LITHOLOGIC LEGEND relating to drill logs.
2. See plate A-12 for GENERAL NOTES relating to drill logs.

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM
DRILL LOGS
OUTLET WORKS
OUTLET PORTAL

SUBMITTED BY: DATE APPROVED:
CHIEF, BRANCH

SPEC. NO. DACW09-___-B-____
DISTRICT FILE NO.

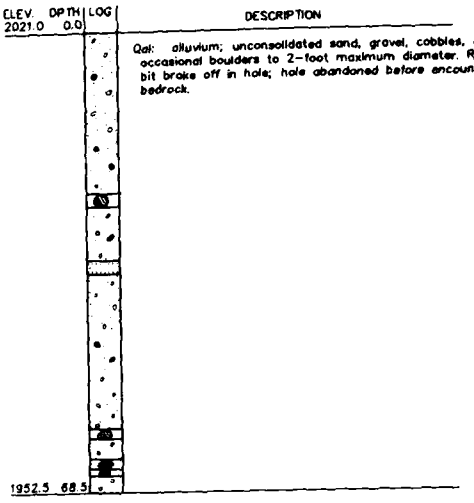
SHEET

PLATE A-23

SAFETY PAYS

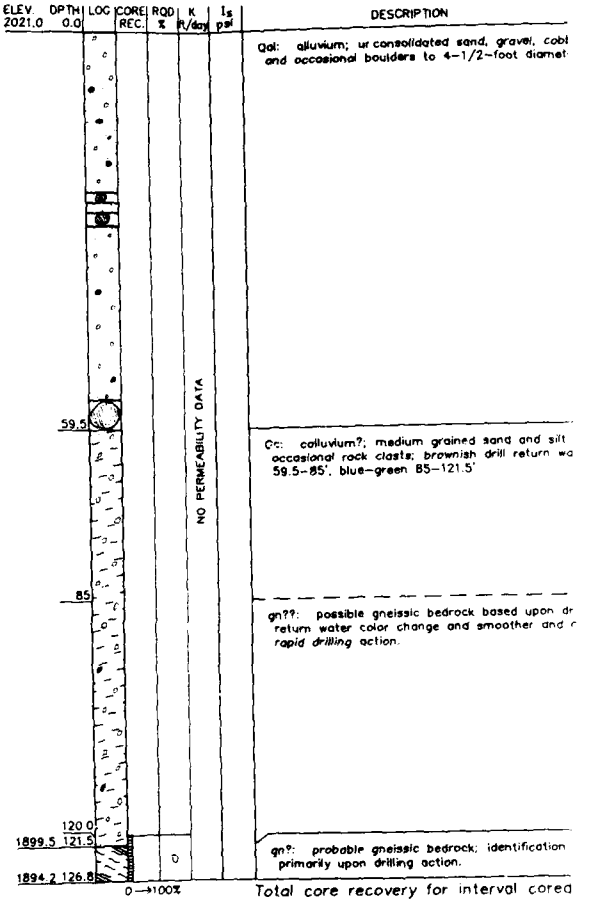
R86-08

N 709390
E 1743675
INCLINATION: VERTICAL



C86-10

N 709390
E 1743685
INCLINATION: VERTICAL



Total core recovery for interval cored
Composite RQD = 0%
Average discontinuities/ft = 10.0

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

C86-10
N 709390
E 1743685
INCLINATION: VERTICAL

RQD %	K ft/day	Is psi	DESCRIPTION
	NO PERMEABILITY DATA		Qal: alluvium; ur consolidated sand, gravel, cobbles, and occasional boulders to 4-1/2-foot diameter.
			Qc: colluvium?; medium grained sand and silt with occasional rock clasts; brownish drill return water 59.5-85', blue-green 85-121.5'.
			gn??: possible gneissic bedrock based upon drill return water color change and smoother and more rapid drilling action.
0			gn?: probable gneissic bedrock; identification based primarily upon drilling action.

→100%

Total core recovery for interval cored = 16%
Composite RQD = 0%
Average discontinuities/ft = 10.0

NOTES:

1. See plate A-11 for LITHOLOGIC LEGEND relating to drill logs.
2. See plate A-12 for GENERAL NOTES relating to drill logs.

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

DRILL LOGS
ENERGY DISSIPATOR

SUBMITTED BY: DATE APPROVED:
CHIEF, ENGINEERING DIVISION

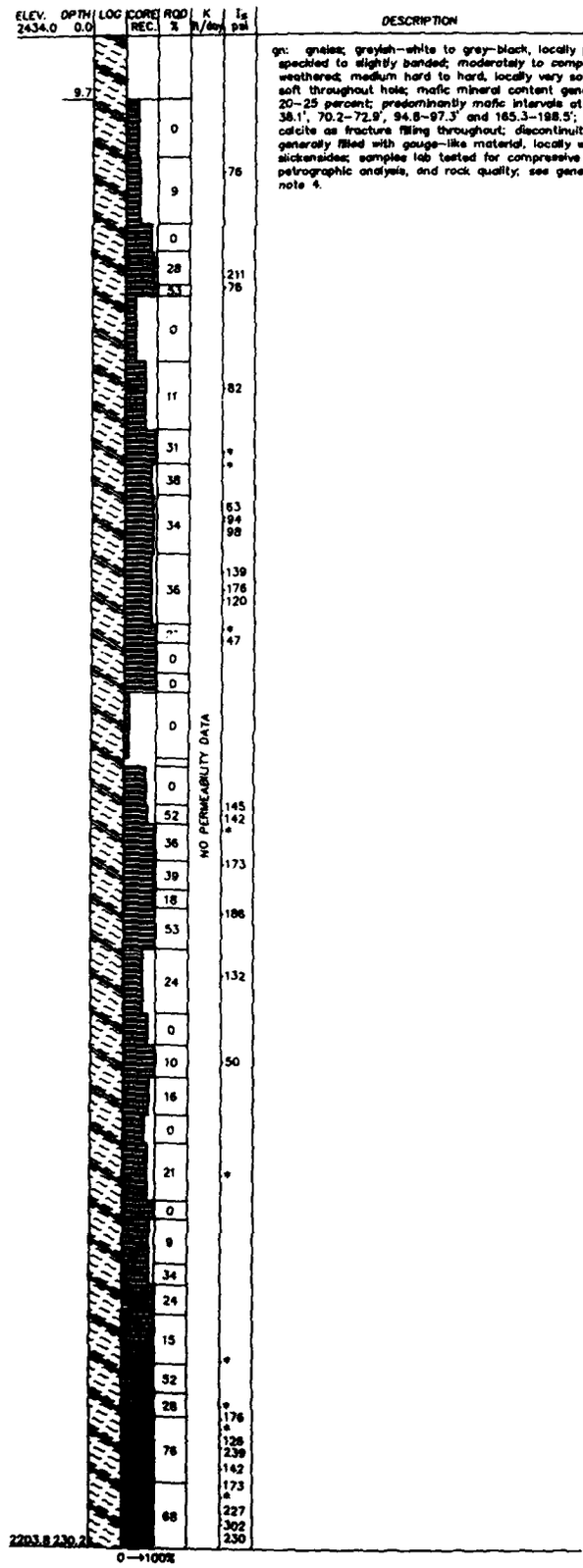
SPEC. NO. DACW09-____-B-____
DISTRICT FILE NO.

SHEET

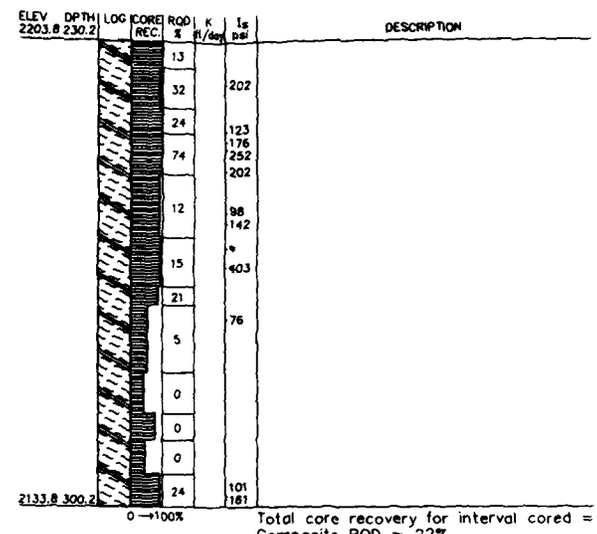
PLATE A-23

C86-09

N 712160
E 1744770
INCLINATION: VERTICAL



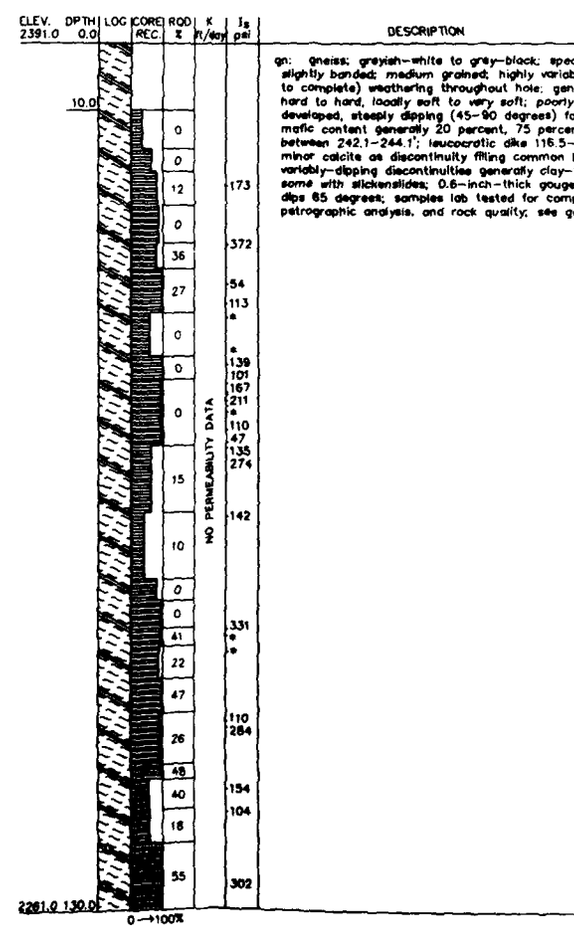
C86-09 (Continued)



Total core recovery for interval cored = 77%
Composite RQD = 22%
Average discontinuities/ft = 5.8

C86-11

N 712055
E 1744570
INCLINATION: VERTICAL



ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

UE ENGINEERING PAYS

C86-09 (Continued)

K ft/day	I_s psi	DESCRIPTION
	202	
	123	
	176	
	252	
	202	
	98	
	142	
	*	
	403	
	76	
	101	
	161	

Total core recovery for interval cored = 77%
Composite RQD = 22%
Average discontinuities/ft = 5.8

C86-11

N 712055
E 1744570
INCLINATION: VERTICAL

RQD %	K ft/day	I_s psi	DESCRIPTION
			gn: gneiss; greyish-white to grey-black; speckled to slightly banded; medium grained; highly variable (slight to complete) weathering throughout hole; generally medium hard to hard, locally soft to very soft; poorly to well-developed, steeply dipping (45-90 degrees) foliation; mafic content generally 20 percent, 75 percent mafics between 242.1-244.1'; leucocratic dike 116.5-119.7'; minor calcite as discontinuity filling common below 38'; variably-dipping discontinuities generally clay-filled, some with slickensides; 0.6-inch-thick gouge zone at 60.7' dips 65 degrees; samples lab tested for compressive strength, petrographic analysis, and rock quality; see general note 4.
0			
0			
12		173	
0			
36		372	
27		54	
		113	
0		*	
		*	
0		139	
0		101	
		167	
0		211	
		*	
		110	
		47	
		135	
15		274	
		142	
10			
0			
0			
61		331	
		*	
		*	
22			
47			
		110	
26		284	
48			
40		154	
		104	
18			
55		302	

NO PERMEABILITY DATA

C86-11 (Continued)

ELEV. 2261.0	DPTH 130.0	LOG	CORE REC.	RQD %	K ft/day	I_s psi	DESCRIPTION
				0			
				0		410	
				12			
				13		*	
				33			
				36			
				12		350	
				50		198	
						205	
						113	
						224	
				73		221	
						148	
						170	
						*	
						173	
				57		117	
						104	
						394	
				0		*	
						*	
						302	
				50		*	
						123	
						274	
				40		*	
				19			
				0		*	
				63			
						419	
				82		*	
				28			
				0			
				14		142	
				27		205	
				65		189	
				0			
2120.6	270.4						

0 → 100%

Total core recovery for interval cored = 85%
Composite RQD = 28%
Average discontinuities/ft = 5.0

NOTES:

1. See plate A-11 for LITHOLOGIC LEGEND relating to drill logs.
2. See plate A-12 for GENERAL NOTES relating to drill logs.
3. High concentrations of point load tests are generally related to intervals laboratory tested for rock quality.

SYMBOL	DESCRIPTION	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

DRILL LOGS
GOVERNMENT CANYON RIDGE

SUBMITTED BY: DATE APPROVED:

SPEC. NO. DACW09- ___ B- ___

DISTRICT FILE NO.

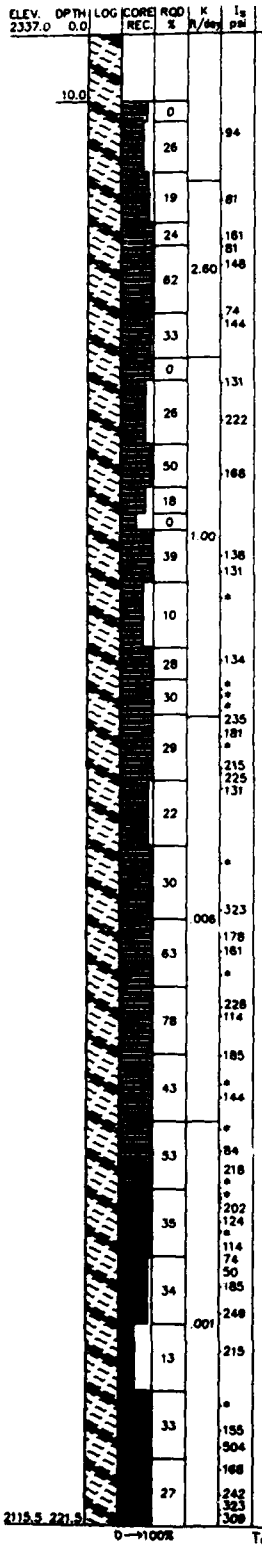
SHEET 1 OF 2 SHEETS

PLATE A-24

SAFETY PAYS

C86-14

N 711960
E 1744450
INCLINATION: VERTICAL



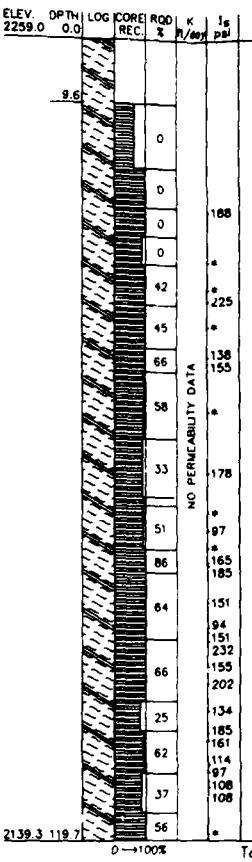
DESCRIPTION

gn: gneiss; greyish-white to grey-black; speckled to distinctly banded; fine to coarse grained; 10-30' generally highly to completely weathered; 30-221.5' generally moderately to slightly weathered, locally highly to completely weathered; generally medium hard, locally hard to very soft; foliation poorly to well-developed and steeply dipping (45-90 degrees); 15-25 percent mafics, except 50 percent mafics from 101.1-102.5' and 181.5-185.6'; leucocratic to pinkish fine grained dikes at 28.9' (0.2-foot-thick, broken), 81.3' (0.9-foot-thick, broken), 154.9' (0.4-foot-thick, dipping 50 degrees), 195.2' (8.8-foot-thick, sheared contact, dipping 75 degrees), 215.4' (0.15-foot-thick, dipping 40 degrees); minor calcite below 38.7'; variably-dipping discontinuities, 40 percent dip 35-45 degrees, generally slightly rough, locally rough to slickensided, generally clay-filled, but locally clean; samples lab tested for petrographic analysis and rock quality; see general note 4.

Total core recovery for interval cored = 89%
Composite RQD = 34%
Average discontinuities/ft = 6.8

C86-27

N 712155
E 1744370
INCLINATION: VERTICAL



DESCRIPTION

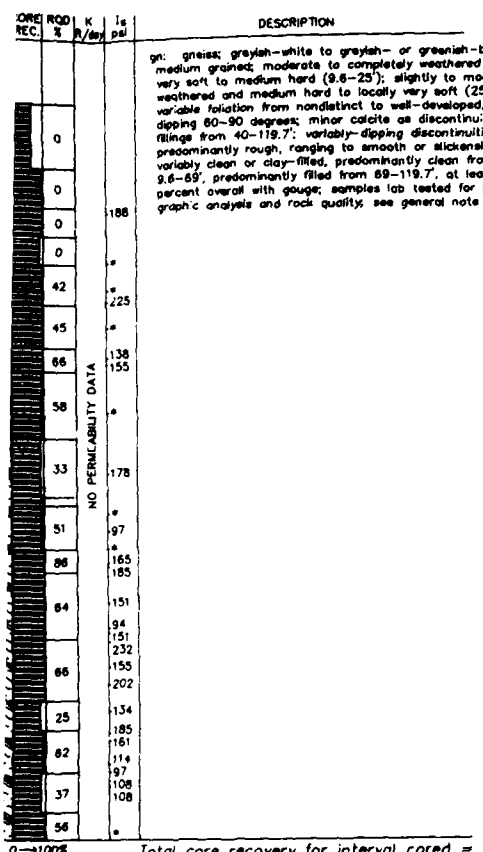
gn: gneiss; greyish-white to greyish- or g
medium grained; moderate to completely
very soft to medium hard (9.6-25'); slight
weathered and medium hard to locally very
variable foliation from nondistinct to well-d
dipping 60-90 degrees; minor calcite as di
fillings from 40-119.7'; variably-dipping dis
predominantly rough, ranging to smooth or
variably clean or clay-filled, predominantly
9.6-69', predominantly filled from 69-119.
percent overall with gouge; samples lab te
graphic analysis and rock qua'ity; see gene

Total core recovery for interval co
Composite RQD = 41%
Average discontinuities/ft = 4.8

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

ALUE ENGINEERING PAYS

C86-27
N 712155
E 1744370
INCLINATION: VERTICAL



Total core recovery for interval cored = 93%
Composite RQD = 41%
Average discontinuities/ft = 4.8

NOTES:

1. See plate A-11 for LITHOLOGIC LEGEND relating to drill logs.
2. See plate A-12 for GENERAL NOTES relating to drill logs.
3. High concentrations of point load tests are generally related to intervals laboratory tested for rock quality.

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

DRILL LOGS
GOVERNMENT CANYON RIDGE

SUBMITTED BY: DATE APPROVED:
CHIEF, ENGINEERING DIVISION

SPEC. NO. DACW09-___-___
DISTRICT FILE NO.

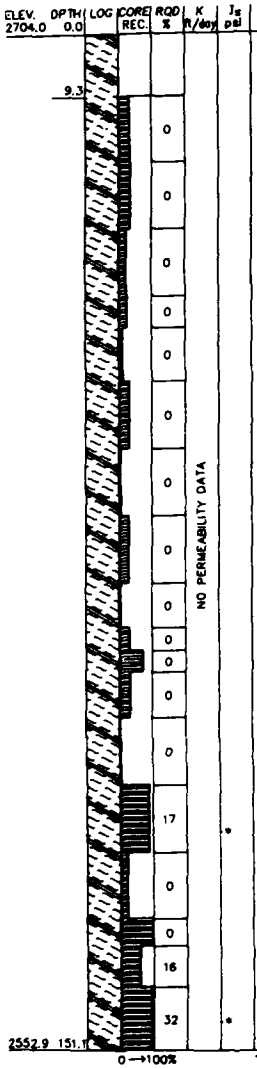
SHEET 2 OF 2 SHEETS

PLATE A-26

SAFETY PAYS

C86-15

N 709258
E 1746180
INCLINATION: VERTICAL

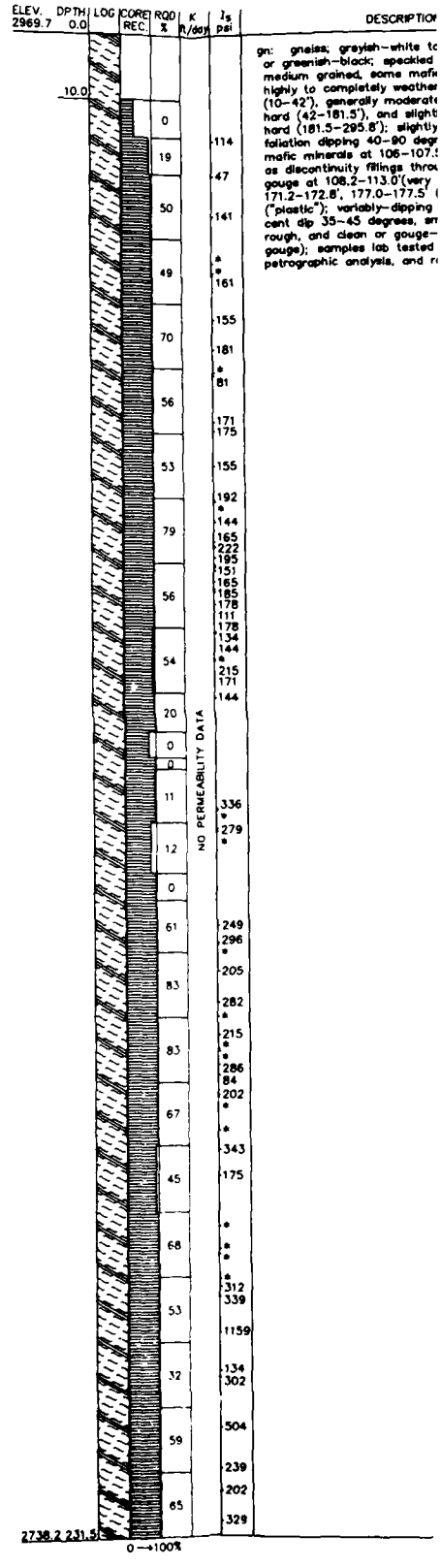


gn: gneiss; brown, light grey to dark grey, and pink; massive to mottled with weak foliation dipping 25–60 degrees; mafic content ranging from 5–20 percent with mafic-rich (40 percent max.) zones 0.2–0.5-foot-thick throughout the core; generally moderately to highly weathered and soft to medium hard, completely weathered and very soft from 9.3–30' and 93–102', discontinuities dip predominantly 60 degrees with a range of 10–90 degrees, about 60 percent with slickensides or gouge; quartzofeldspathic dike dipping near 80 degrees at 139'; gouge zones at 74' (0.3-foot-thick) and 122' (0.25-inch-thick) with chloritic alteration; no samples lab tested.

Total core recovery for interval cored = 37%
Composite RQD = 4%
Average discontinuities/ft = 6.7

C86-16

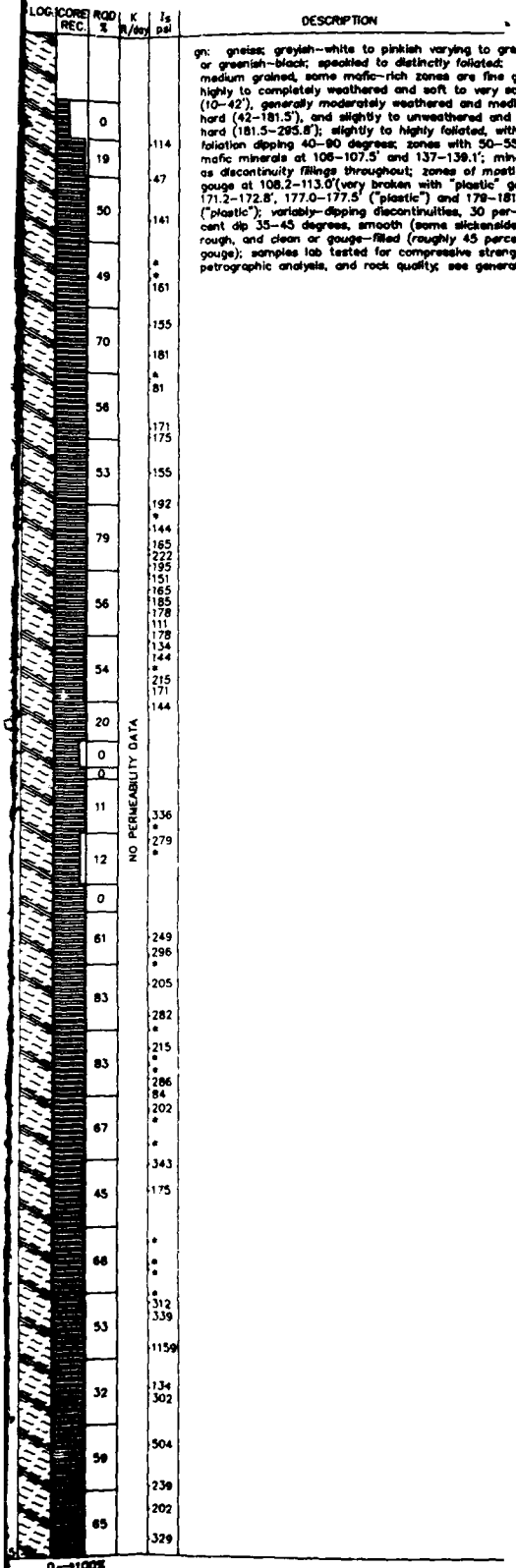
N 709195
E 1746880
INCLINATION: VERTICAL



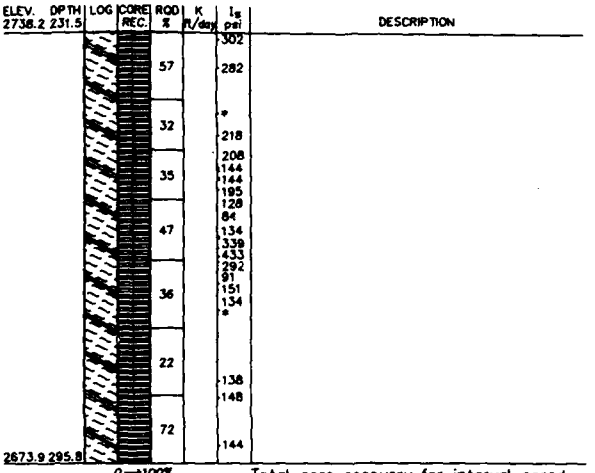
gn: gneiss; greyish-white to or greenish-black; speckled medium grained, some mafic highly to completely weather (10–42'), generally moderate hard (42–181.5'), and slight hard (181.5–295.8'); slightly foliation dipping 40–90 degr mafic minerals at 106–107.5 as discontinuity fillings throu gouge at 108.2–113.0'(very 171.2–172.8', 177.0–177.5' ("plastic"); variably-dipping cent dip 35–45 degrees, sm rough, and clean or gouge-gouge); samples lab tested petrographic analysis, and r

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

C86-16
N 709195
E 1746880
INCLINATION: VERTICAL



C86-16 (Continued)



Total core recovery for interval cored = 97%
Composite RQD = 49%
Average discontinuities/ft = 6.3

NOTES:

1. See plate A-11 for LITHOLOGIC LEGEND relating to drill logs.
2. See plate A-12 for GENERAL NOTES relating to drill logs.
3. High concentrations of point load tests are generally related to intervals laboratory tested for rock quality.

SYMBOL	DESCRIPTION	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

DRILL LOGS
SPILLWAY

SUBMITTED BY: DATE APPROVED:
CHIEF, ENGINEERING DIVISION

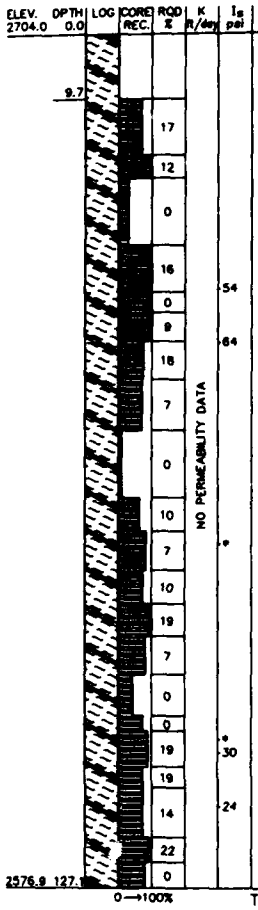
SPEC. NO. DACW09- - -
DISTRICT FILE NO.

SHEET 1 OF 4 SHEETS

PLATE A-20

C86-17

N 709255
E 1746212
INCLINATION: VERTICAL



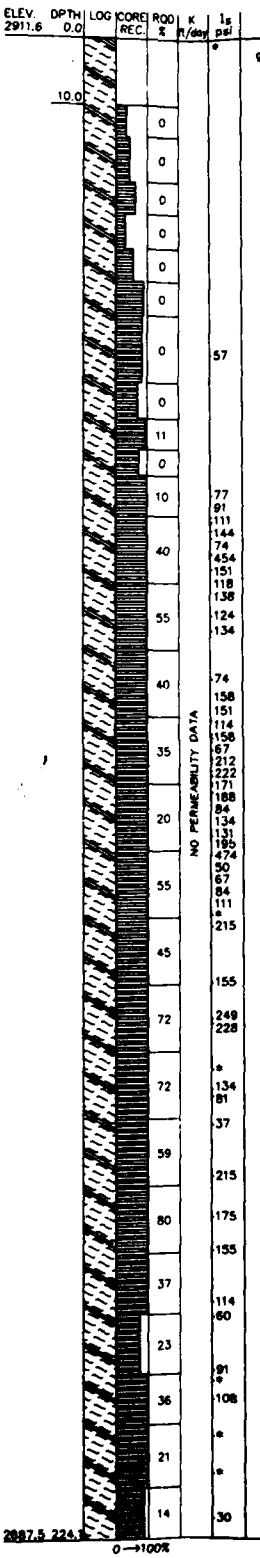
DESCRIPTION

gn: gneiss; brown, light grey to dark grey, occasionally pink; massive, weakly- to well-foliated with foliations dipping 30 degrees and 60-80 degrees; highly to completely weathered and soft to medium hard; 2- to 5-foot-thick mafic-rich (15-30 percent) zones intermittent throughout the core; discontinuities dip predominantly 40-75 degrees, about 60 percent with slickensides or gouge, many healed, seven with about 0.25-inch-wide, gouge-filled apertures; greenish-grey dike (0.2-foot-wide, dipping 50 degrees) at 78.3'; samples lab tested for petrographic analysis and rock quality; see general note 4.

Total core recovery for interval cored = 69%
Composite RQD = 9%
Average discontinuities/ft = 4.8

C86-18

N 709015
E 1746520
INCLINATION: VERTICAL

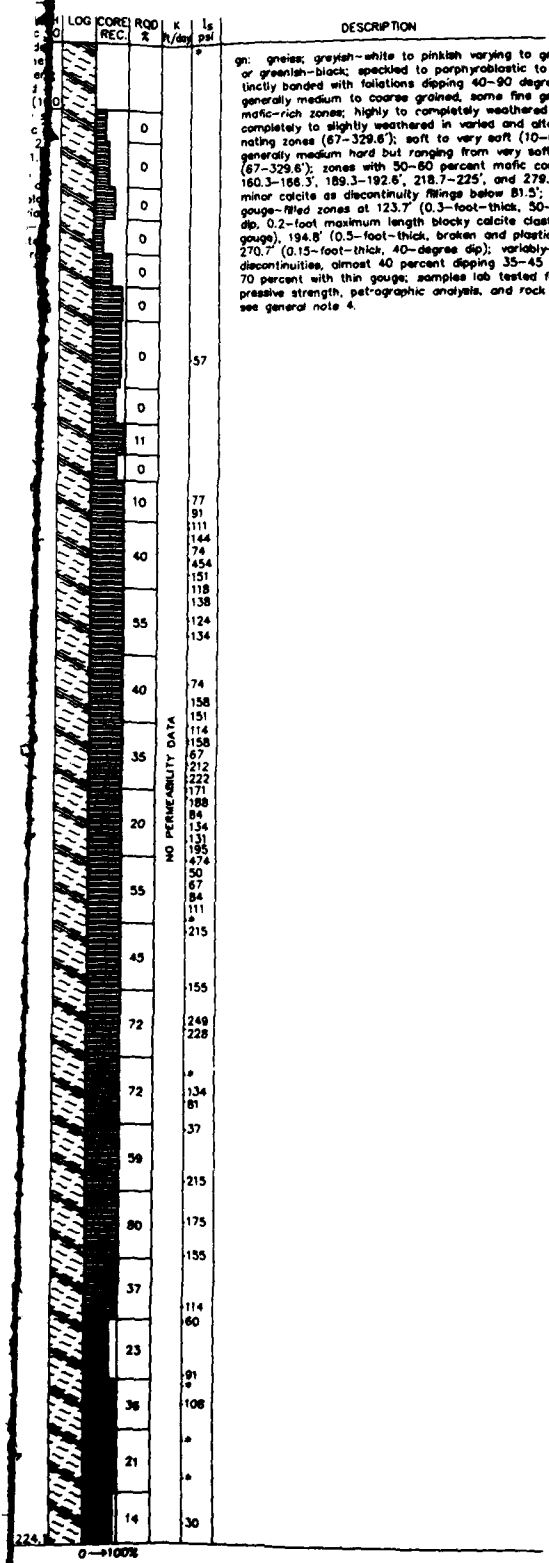


DESCRIPTION

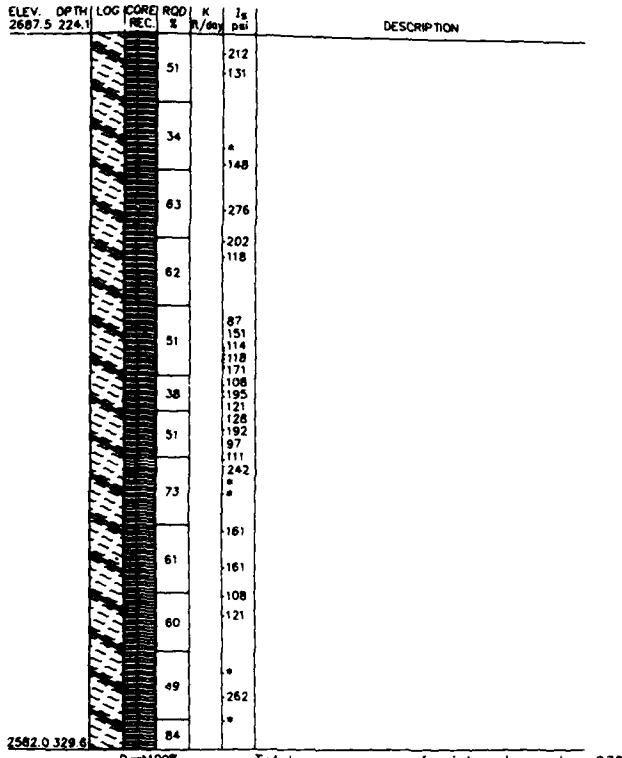
gn: gneiss; greyish-white to pinkish or greenish-black; speckled to porp tinctly banded with foliations dippin generally medium to coarse grained mafic-rich zones; highly to complet completely to slightly weathered in nating zones (67-328.6'); soft to generally medium hard but ranging (67-328.6'); zones with 50-60 per 160.3-166.3', 189.3-192.6', 218.7-2 minor calcite as discontinuity fillings gouge-filled zones at 123.7' (0.3-f dip, 0.2-foot maximum length block gouge), 194.8' (0.5-foot-thick, bro 270.7' (0.15-foot-thick, 40-degree discontinuities, almost 40 percent o 70 percent with thin gouge; sample pressive strength, petrographic anal see general note 4.

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

C86-18
N 709015
E 1746520
INCLINATION: VERTICAL



C86-18 (Continued)



Total core recovery for interval cored = 93%
Composite RQD = 40%
Average discontinuities/ft = 6.2

NOTES:

1. See plate A-11 for LITHOLOGIC LEGEND relating to drill logs.
2. See plate A-12 for GENERAL NOTES relating to drill logs.
3. High concentrations of point load tests are generally related to intervals laboratory tested for rock quality.

SYMBOL	DESCRIPTION	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

DRILL LOGS
SPILLWAY

SUBMITTED BY: DATE APPROVED:
CHIEF, ENGR. DIV.

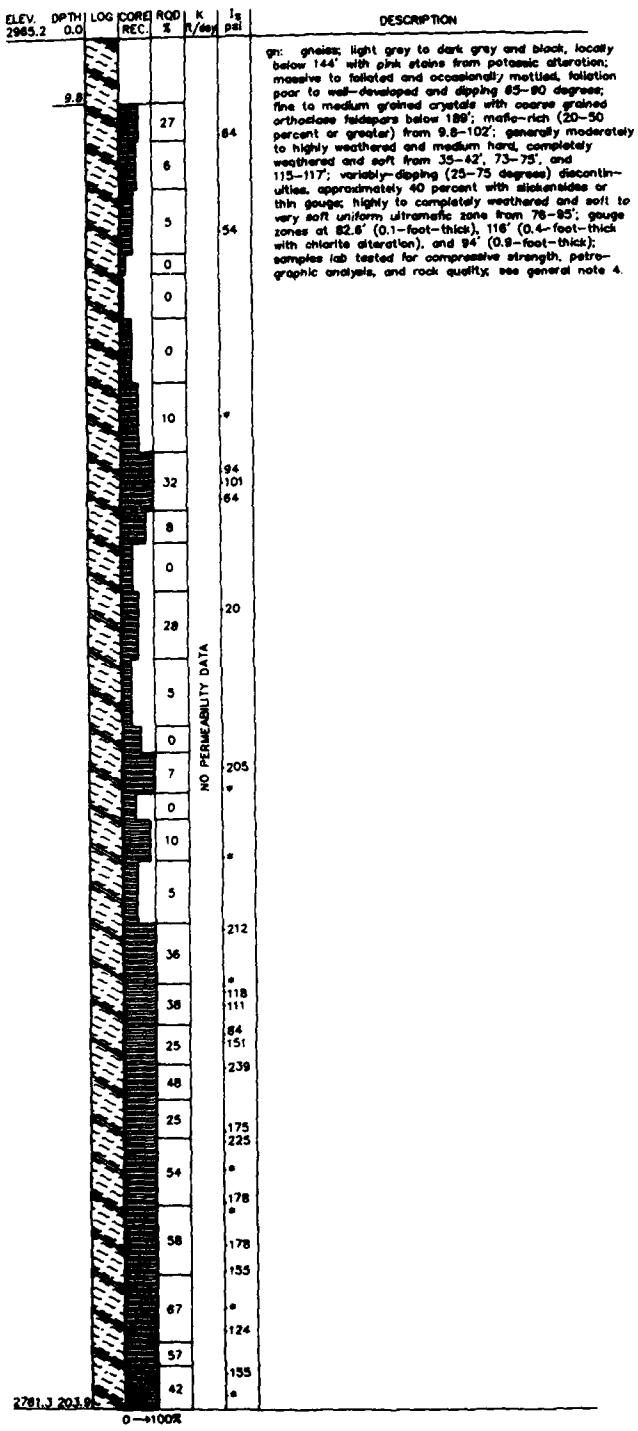
SPEC. NO. DACW09-____-R-____
DISTRICT FILE NO.

SHEET 2 OF 4 SHEETS

PLATE A-27

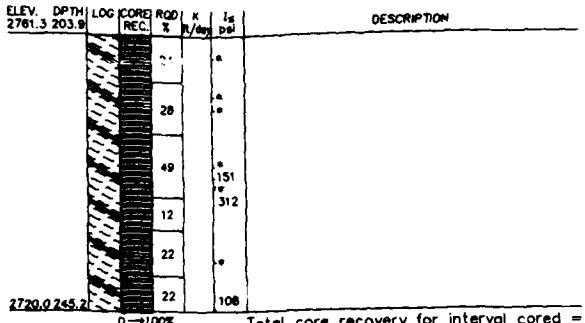
C86-19

N 708910
E 1746060
INCLINATION: VERTICAL



gn: gneiss; light grey to dark grey and black, locally below 144' with pink stains from potassic alteration; massive to foliated and occasionally mottled, foliation poor to well-developed and dipping 65-90 degrees; fine to medium grained crystals with coarse grained orthoclase feldspars below 188'; mafic-rich (20-50 percent or greater) from 9.8-102'; generally moderately to highly weathered and medium hard, completely weathered and soft from 35-42', 73-75', and 115-117'; variably-dipping (25-75 degrees) discontinuities, approximately 40 percent with slickensides or thin gouge; highly to completely weathered and soft to very soft uniform ultramafic zone from 76-95'; gouge zones at 82.6' (0.1-foot-thick), 116' (0.4-foot-thick with chlorite alteration), and 94' (0.9-foot-thick); samples lab tested for compressive strength, petrographic analysis, and rock quality; see general note 4.

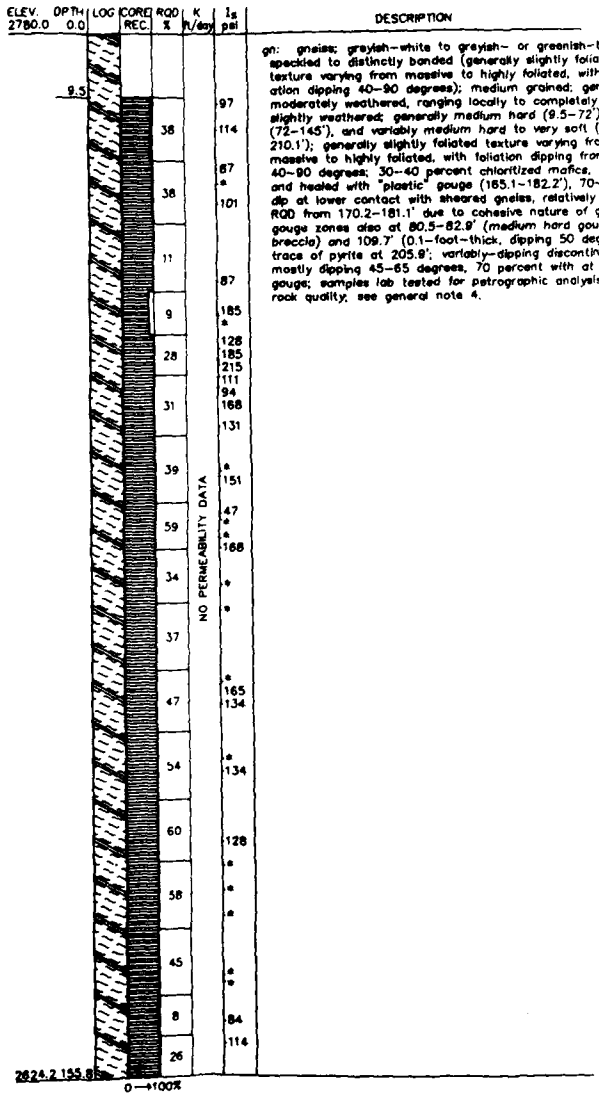
C86-19 (Continued)



Total core recovery for interval cored = 75%
Composite RQD = 24%
Average discontinuities/ft = 5.9

C86-20

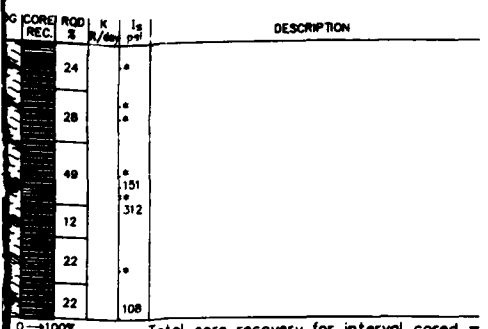
N 708795
E 1746635
INCLINATION: VERTICAL



gn: gneiss; greyish-white to greyish- or greenish-black speckled to distinctly banded (generally slightly foliated texture varying from massive to highly foliated, with foliation dipping 40-90 degrees); medium grained; generally moderately weathered, ranging locally to completely or slightly weathered; generally medium hard (9.5-72'), hard (72-145'), and variably medium hard to very soft (145-210.1'); generally slightly foliated texture varying from massive to highly foliated, with foliation dipping from 40-90 degrees; 30-40 percent chloritized mafics, sheared and healed with "plastic" gouge (185.1-182.2'), 70-deg dip at lower contact with sheared gneiss, relatively high RQD from 170.2-181.1' due to cohesive nature of gouge; gouge zones also at 80.5-82.9' (medium hard gouge and breccia) and 109.7' (0.1-foot-thick, dipping 50 degrees); trace of pyrite at 205.9'; variably-dipping discontinuities, mostly dipping 45-65 degrees, 70 percent with at least gouge; samples lab tested for petrographic analysis and rock quality; see general note 4.

ENHANCEMENT THRU ENGINEERING

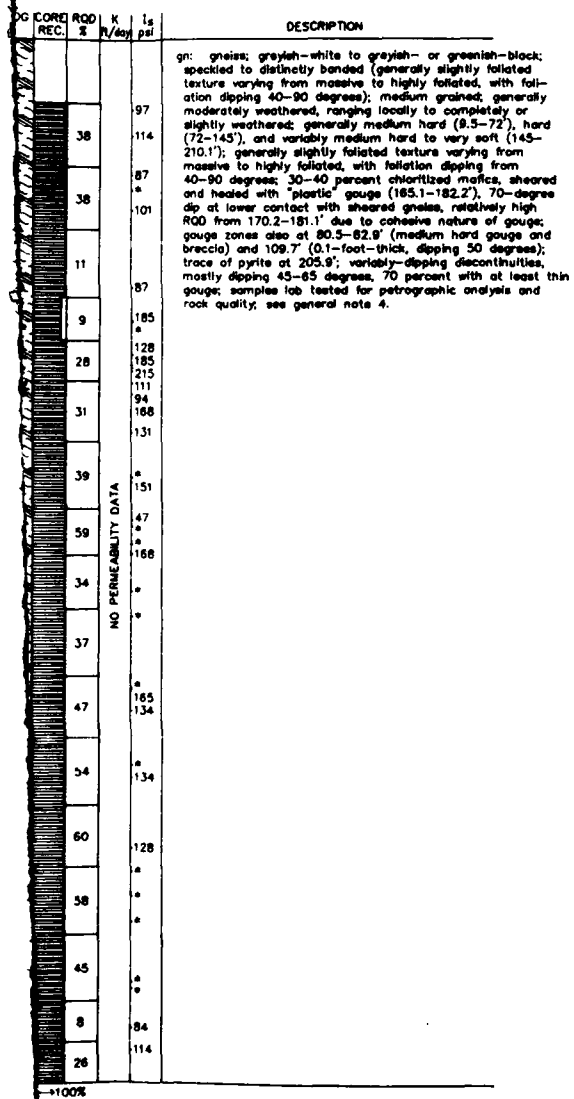
C86-19 (Continued)



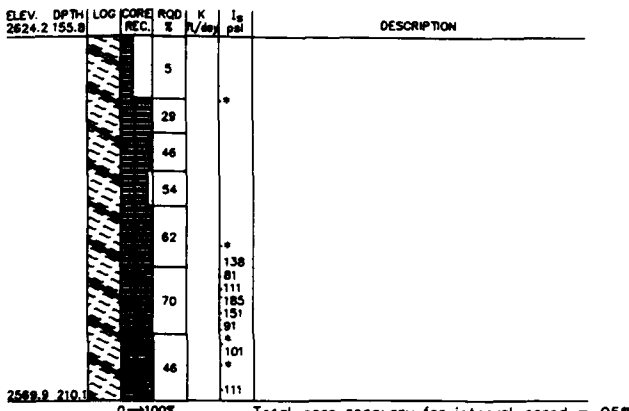
Total core recovery for interval cored = 75%
Composite RQD = 24%
Average discontinuities/ft = 5.9

C86-20

N 708795
E 1746635
INCLINATION: VERTICAL



C86-20 (Continued)



Total core recovery for interval cored = 95%
Composite RQD = 40%
Average discontinuities/ft = 7.1

NOTES:

1. See plate A-11 for LITHOLOGIC LEGEND relating to drill logs.
2. See plate A-12 for GENERAL NOTES relating to drill logs.
3. High concentrations of point load tests are generally related to intervals laboratory tested for rock quality.

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

DRILL LOGS
SPILLWAY

SUBMITTED BY: DATE APPROVED:
CHIEF, ENGR. DIVISION

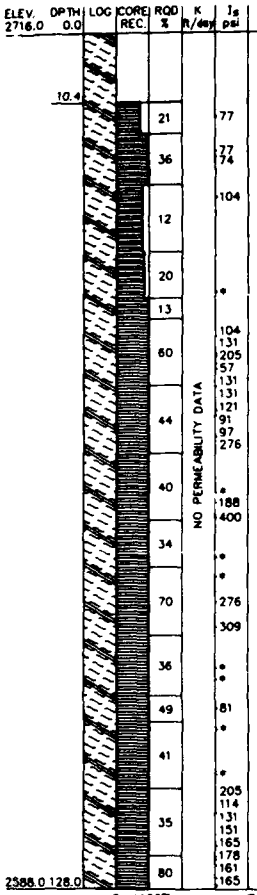
SPEC. NO. DACW09-____-B-____
DISTRICT FILE NO.

SHEET 3 OF 4 SHEETS

PLATE A-28

C86-21

N 708620
E 1746750
INCLINATION: VERTICAL



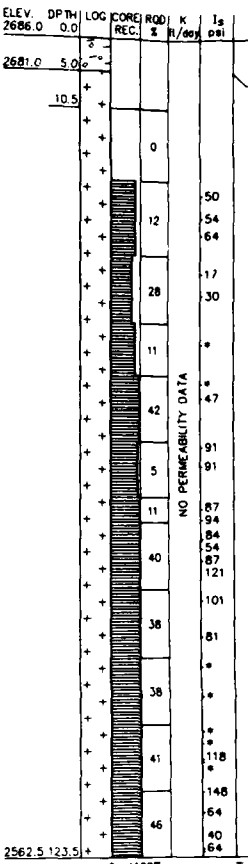
DESCRIPTION

gn: gneiss; brown, light grey to dark grey, and black; massive, mottled to well-foliated with foliations dipping 25–30 degrees and 50–60 degrees; very fine to medium grained, small cluster of pyrite crystals from 94.5–101.5'; mafic content ranging from 5–50 percent and greater; mafic-rich (35–50 percent) zones common; slightly to highly weathered and soft to moderately hard; discontinuities dip predominantly 30–60 degrees, 62 percent with slickensides or gouge; quartzofeldspathic dike (0.1-foot-wide, dipping 60 degrees) at 65'; calcite-filled fractures at 89.5', 91.5', and 110'; samples lab tested for compressive strength, petrographic analysis, and rock quality; see general note 4.

Total core recovery for interval cored = 96%
Composite RQD = 40%
Average discontinuities/ft = 5.8

C86-22

N 709600
E 1746640
INCLINATION: VERTICAL



DESCRIPTION

Qc: colluvium; abundant diorite rock clasts in a silty sand matrix.

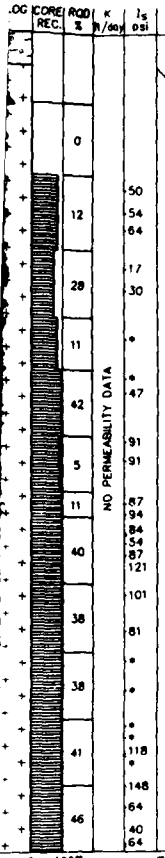
qd: diorite; brownish-grey and medium grey; sp generally massive; medium to coarse grained, n to completely weathered and soft to very soft, tinuities dip predominantly 40–75 degrees, 65 p with slickensides or gouge; two transitional zone gneissic foliation dipping 45–50 degrees at 61.0 and 104.3–110.3'; samples lab tested for petrog analysis and rock quality; see general note 4

Total core recovery for interval cored
Composite RQD = 27%
Average discontinuities/ft = 5.0

ENVIRONMENTAL
ENHANCEMENT
THRU ENGINEERING

C86-22

N 709600
E 1746640
INCLINATION: VERTICAL



Qc: colluvium; abundant diorite rock clasts in a slightly silty sand matrix.

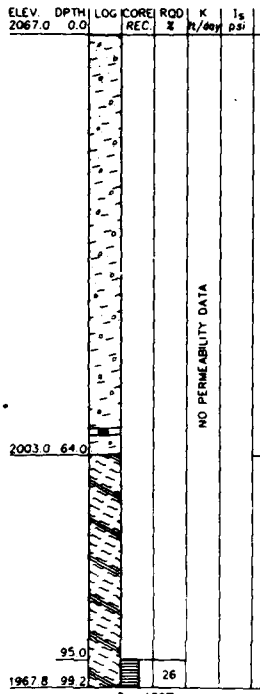
qd: diorite; brownish-grey and medium grey; speckled; generally massive; medium to coarse grained; moderately to completely weathered and soft to very soft; discontinuities dip predominantly 40-75 degrees, 65 percent with slickensides or gouge; two transitional zones of gneissic foliation dipping 45-50 degrees at 81.0-81.5' and 104.3-110.3'; samples lab tested for petrographic analysis and rock quality; see general note 4.

Total core recovery for interval cored = 84%
Composite RQD = 27%
Average discontinuities/ft = 5.0

AUXILIARY SPILLWAY

C84-09

N 709110
E 1743970
INCLINATION: VERTICAL



Qc: colluvium; unconsolidated slightly silty sand, sand, and minor gravel; one boulder from 60-61'.

gn: gneiss, light greyish-white with minor speckles of dark grey and black; slightly weathered and hard; quartzofeldspathic (mafic content about 5 percent); highly fractured; description based upon 2.4' of recovered core.

Total core recovery for interval cored = 57%
Composite RQD = 0%
Average discontinuities/ft = 4.2

NOTES:

1. See plate A-11 for LITHOLOGIC LEGEND relating to drill logs.
2. See plate A-12 for GENERAL NOTES relating to drill logs.
3. High concentrations of point load tests are generally related to intervals laboratory tested for rock quality.

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

DRILL LOGS
SPILLWAY

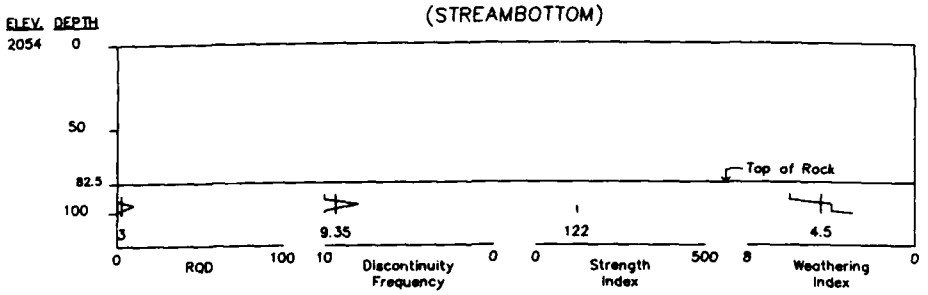
SUBMITTED BY: DATE APPROVED:
CHIEF, ENGINEERING DIVISION

SPEC. NO DACW07-____-8-____
DISTRICT FILE NO.

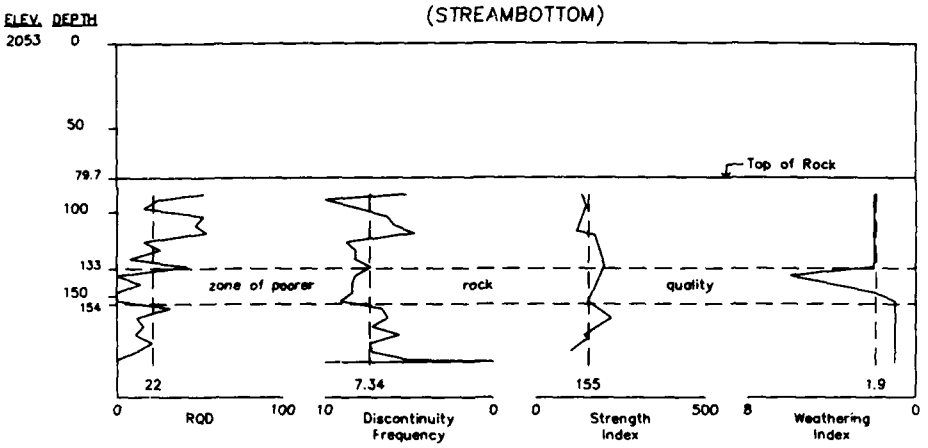
SHEET 4 OF 4 SHEETS

PLATE A-28

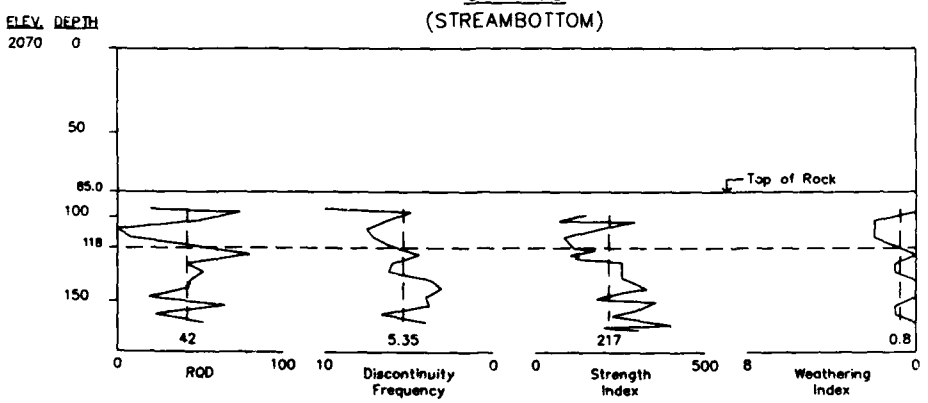
C84-01
(STREAMBOTTOM)



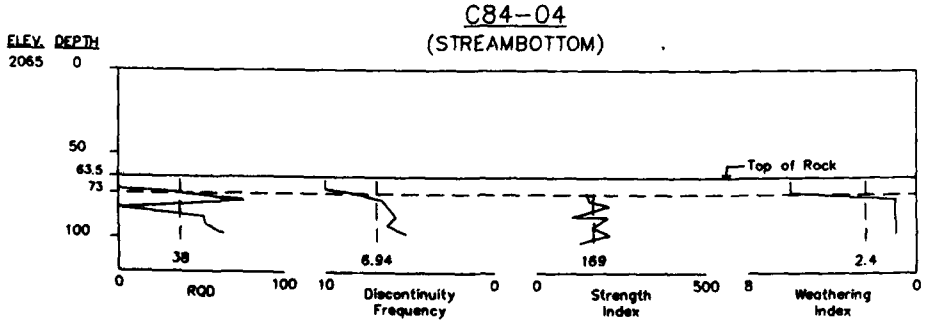
C84-02
(STREAMBOTTOM)



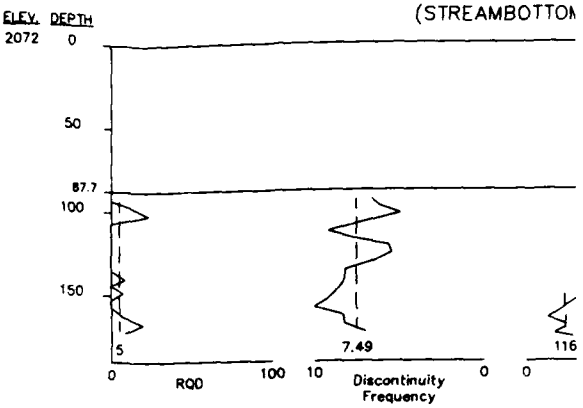
C84-03
(STREAMBOTTOM)



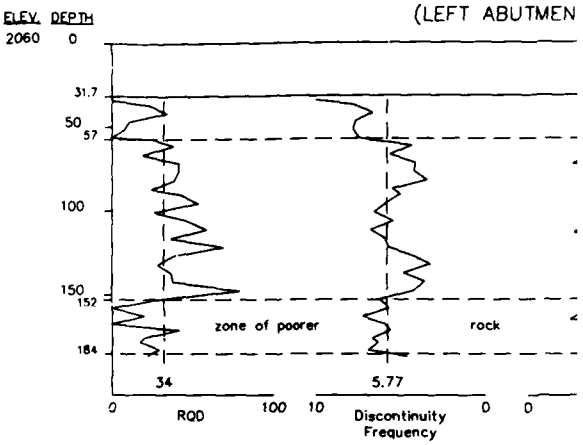
C84-04
(STREAMBOTTOM)



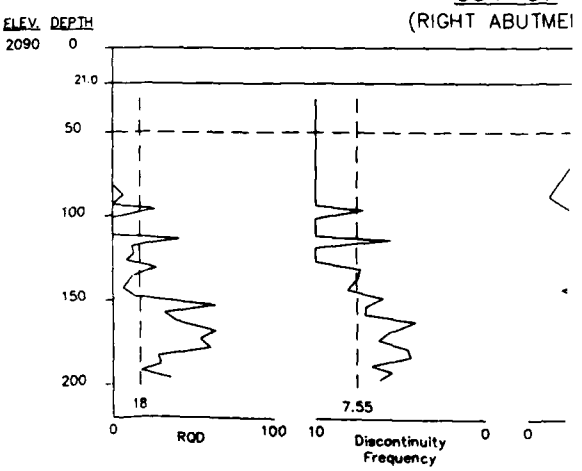
C84-05
(STREAMBOTTOM)



C84-06
(LEFT ABUTMEN

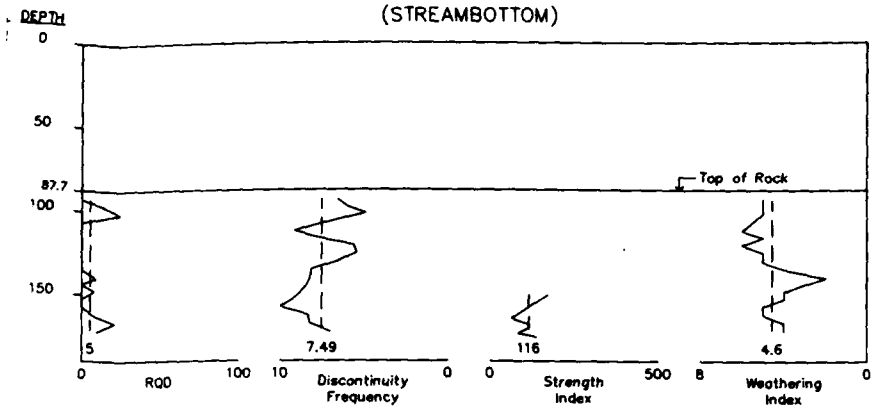


C84-07
(RIGHT ABUTMEI

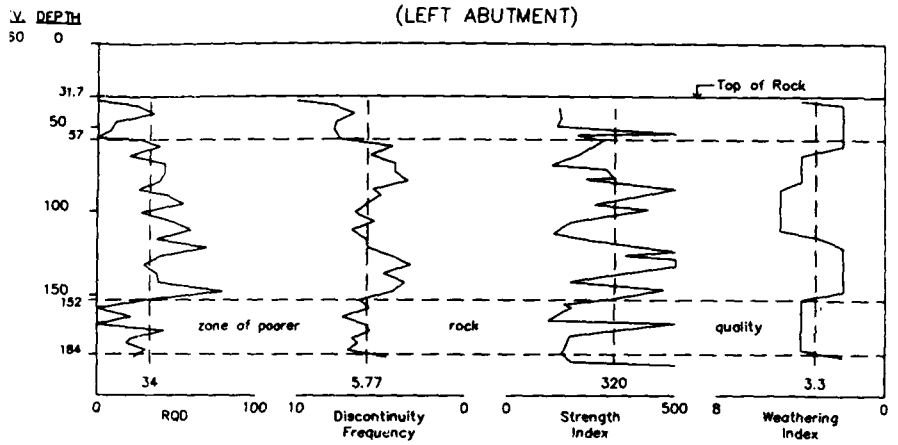


ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

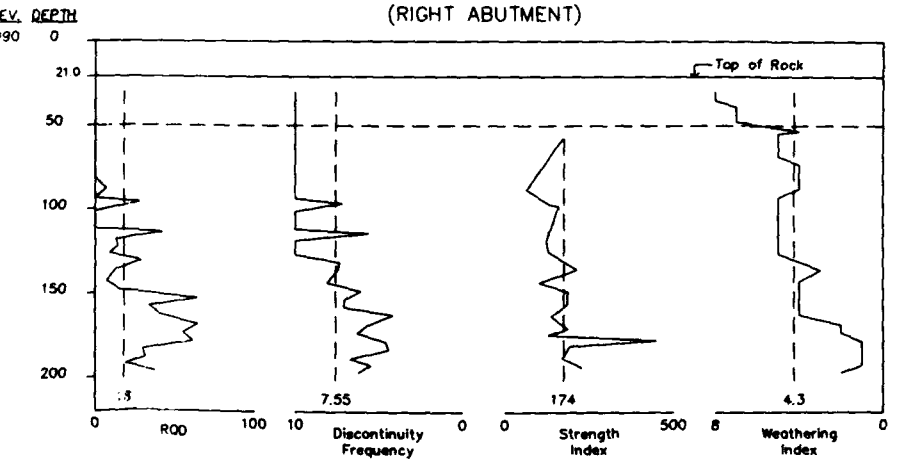
C84-05
(STREAMBOTTOM)



C84-06
(LEFT ABUTMENT)



C84-07
(RIGHT ABUTMENT)



EXPLANATION

RQD — The total length of intact core pieces greater than 4 inches in length divided by the length of each core run. The RQD (Rock Quality Designation) is expressed as a percentage.

Discontinuity Frequency — The number of incipient or broken discontinuities (fractures, joints, shears, etc.) per foot of recovered core. A value of 10.00 is arbitrarily considered to be the maximum.

Strength Index — The results of point load tests with size corrected to represent 50mm core. As a rough approximation, the unconfined compressive strength is about 20 times greater than the point load strength index. Results are expressed in psi. Strength index values over 500 psi (very hard rock) are plotted as 500.

Weathering Index — An arbitrary numerical scale of core sample weathering based upon field inspection using Bieniawski's (1979) classification method.

Value	Weathering Description
0	Unweathered
1	Unweathered to Slightly Weathered
2	Slightly Weathered
3	Slightly to Moderately Weathered
4	Moderately Weathered or Slightly to Highly Weathered
5	Moderately to Highly Weathered
6	Highly Weathered or Moderately to Completely Weathered
7	Highly to Completely Weathered
8	Completely Weathered

GENERAL NOTES

1. See plate A-9 for locations of drill holes.
2. See plates A-42 thru A-46 for geologic cross sections showing depth to improved quality rock (thickness of the surficial zone of poorer rock quality) and deeper intervals of poorer rock quality.
3. The vertical dashed line through each graph is the graphical representation of the average value of that parameter for that particular core hole. The numerical average is shown at the bottom of the line. Graph deflections to the left indicate poorer than average values. Generally, where all graphed parameters deflect to the left, the interval is identified as a zone of poorer rock quality relative to the overall rock quality in that particular core hole.
4. The upper horizontal dashed line indicates the depth extent of the surficial zone of poorer rock quality relative to the overall rock quality in that particular core hole. Determination is based subjectively and qualitatively. Where no line is shown, rock quality differential was determined to be insignificant or questionable. Deeper zones of poorer rock quality are also identified by dashed horizontal lines.
5. See plates A-11 thru A-29 for logs of drill holes.
6. See section 5 of this appendix under CORE DRILLING for a discussion of how the rock quality parameters were calculated and for a description of the weathering classifications.

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

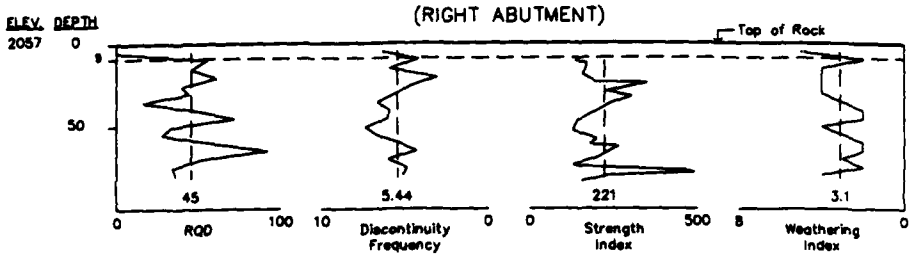
SEVEN OAKS DAM

ROCK QUALITY PARAMETER COMPARISONS

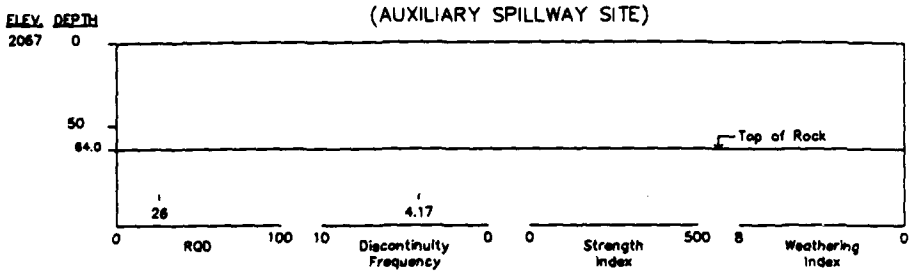
DESIGNED BY:
DRAWN BY:
CHECKED BY:
SUBMITTED BY:
DATE APPROVED:
SPEC. NO. DACW09- - -
DISTRICT FILE NO.
SHEET 1 OF 6 SHEETS

PLATE A-30

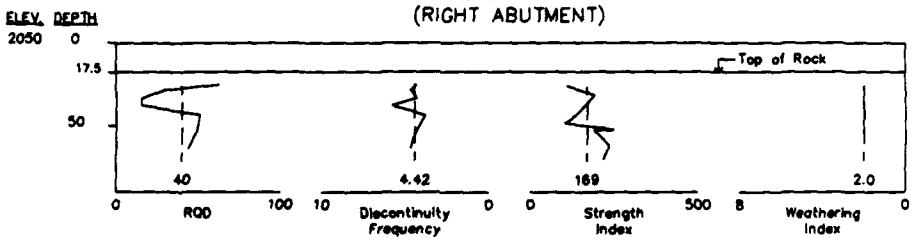
C84-08
(RIGHT ABUTMENT)



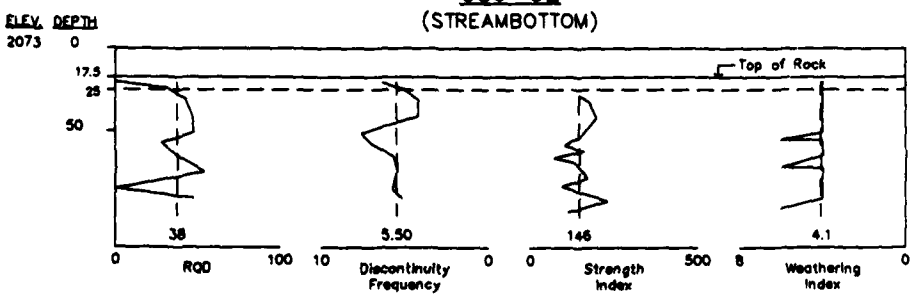
C84-09
(AUXILIARY SPILLWAY SITE)



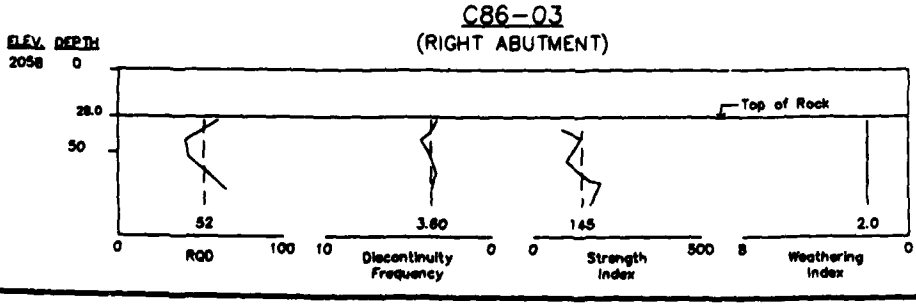
C86-01
(RIGHT ABUTMENT)

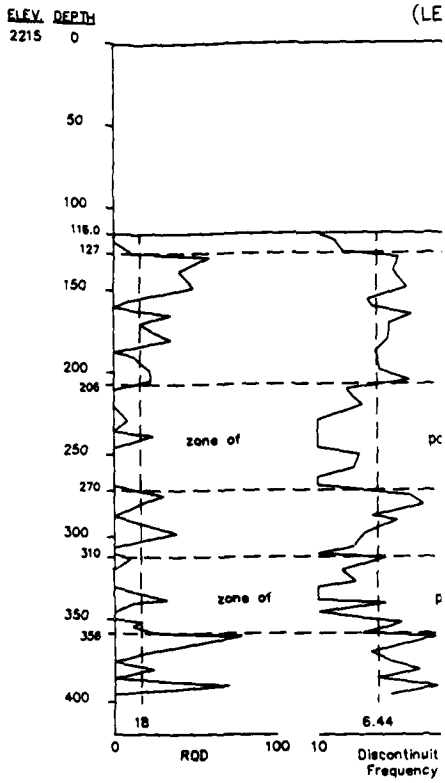


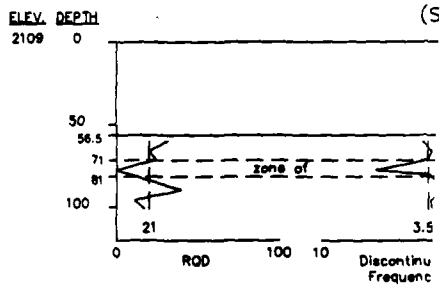
C86-02
(STREAMBOTTOM)

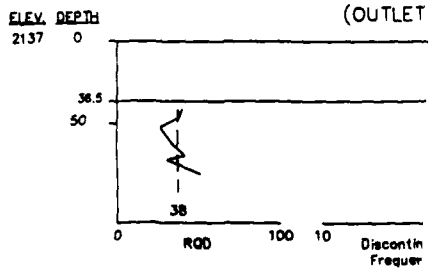


C86-03
(RIGHT ABUTMENT)



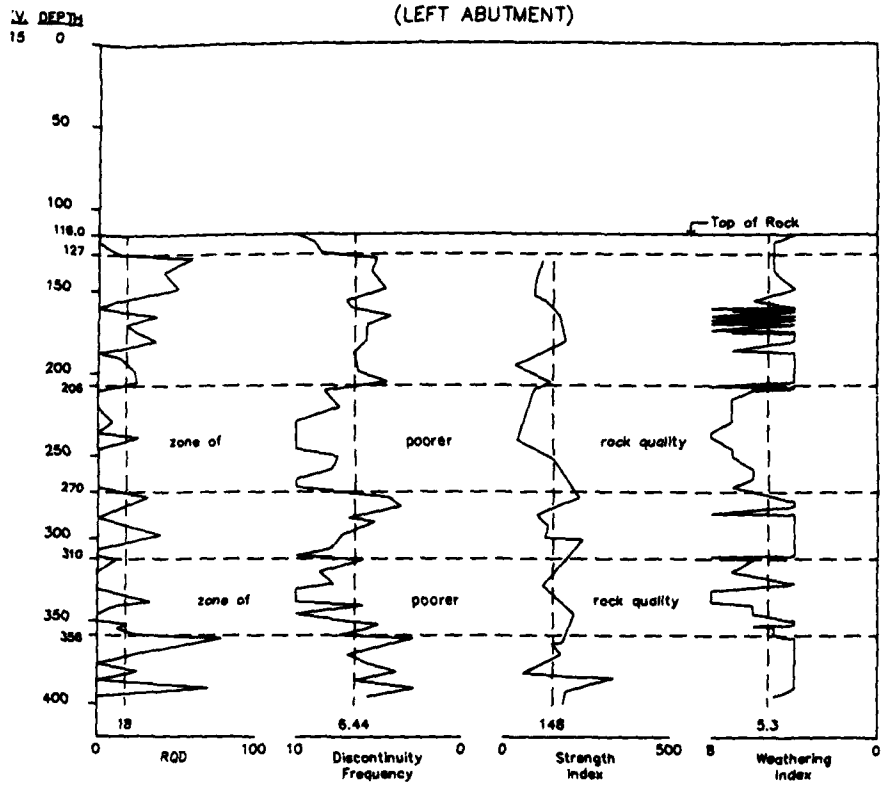




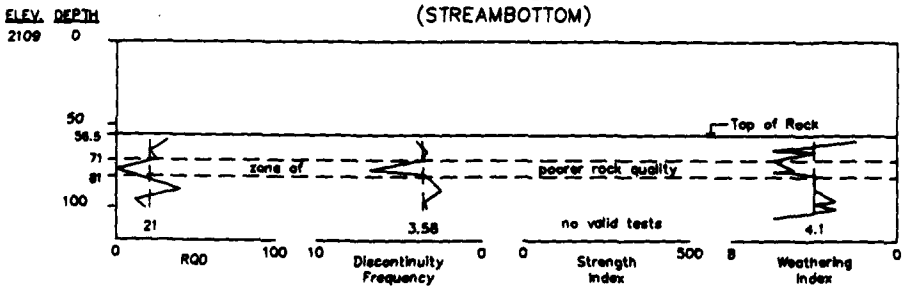


ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

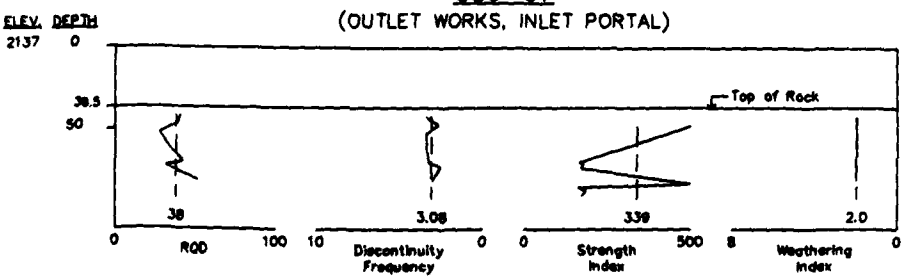
C86-04
(LEFT ABUTMENT)



C86-05
(STREAMBOTTOM)



C86-07
(OUTLET WORKS, INLET PORTAL)



EXPLANATION

RQD — The total length of intact core pieces greater than 4 inches in length divided by the length of each core run. The RQD (Rock Quality Designation) is expressed as a percentage.

Discontinuity Frequency — The number of incipient or broken discontinuities (fractures, joints, shears, etc.) per foot of recovered core. A value of 10.00 is arbitrarily considered to be the maximum.

Strength Index — The results of point load tests with size corrected to represent 50mm core. As a rough approximation, the unconfined compressive strength is about 20 times greater than the point load strength index. Results are expressed in psi. Strength index values over 500 psi (very hard rock) are plotted as 500.

Weathering Index — An arbitrary numerical scale of core sample weathering based upon field inspection using Bieniawski's (1979) classification method.

Value	Weathering Description
0	Unweathered
1	Unweathered to Slightly Weathered
2	Slightly Weathered
3	Slightly to Moderately Weathered
4	Moderately Weathered or Slightly to Highly Weathered
5	Moderately to Highly Weathered
6	Highly Weathered or Moderately to Completely Weathered
7	Highly to Completely Weathered
8	Completely Weathered

NOTE:

See plate A-30 for general notes.

SYMBOL	DESCRIPTION	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY: SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

DRAWN BY:

CHECKED BY:

SEVEN OAKS DAM

ROCK QUALITY PARAMETER COMPARISONS

SUBMITTED BY: DATE APPROVED:

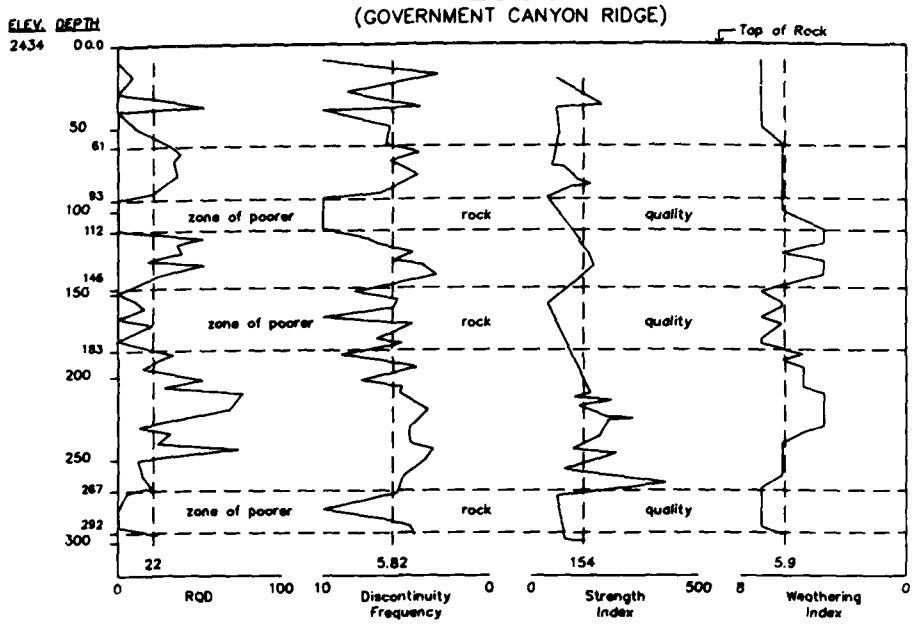
SPEC. NO. SACW 09- ____ -____

DISTRICT FILE NO.

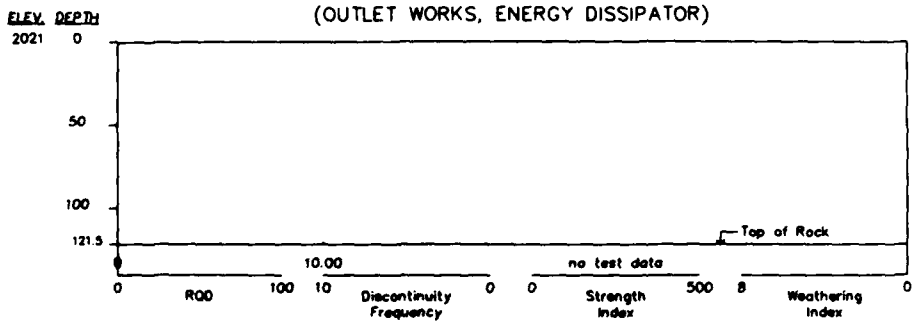
SHEET 2 OF 6 SHEETS

PLATE A-31

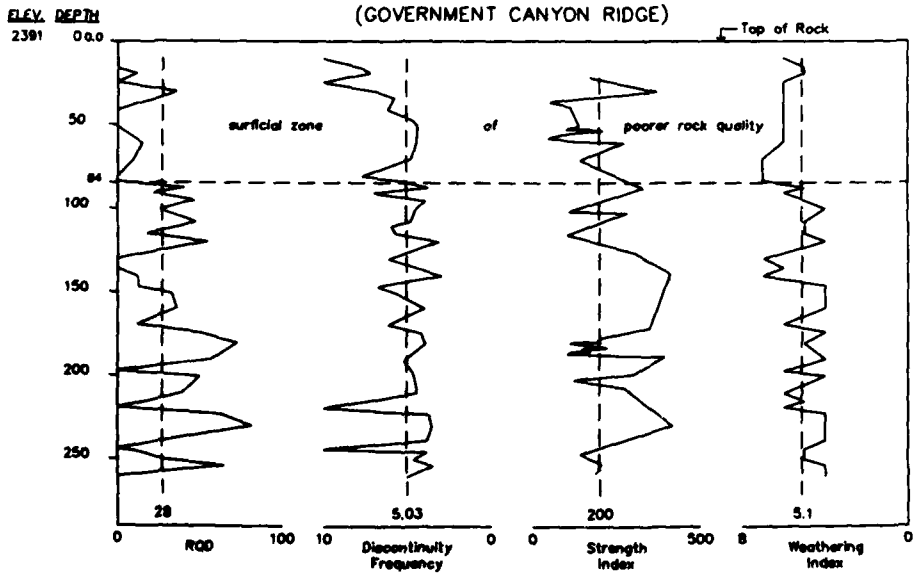
C86-09
(GOVERNMENT CANYON RIDGE)



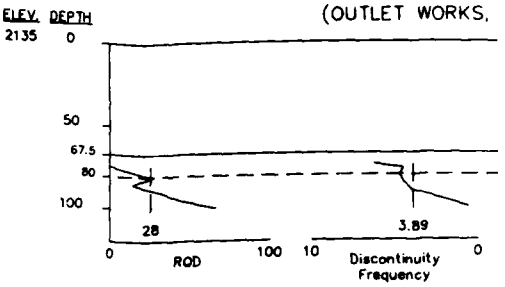
C86-10
(OUTLET WORKS, ENERGY DISSIPATOR)



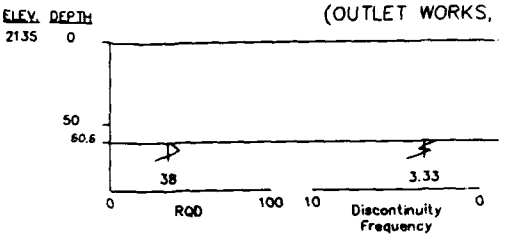
C86-11
(GOVERNMENT CANYON RIDGE)



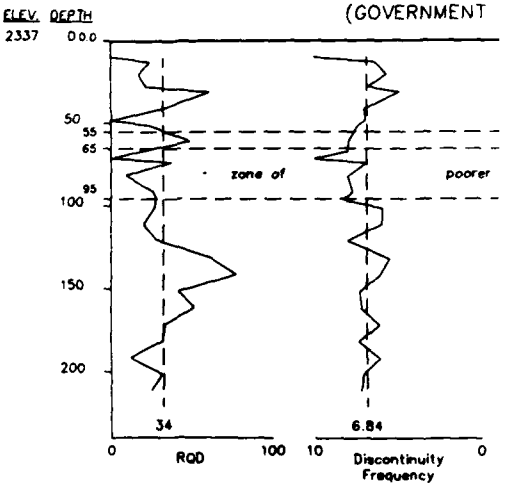
C86
(OUTLET WORKS,



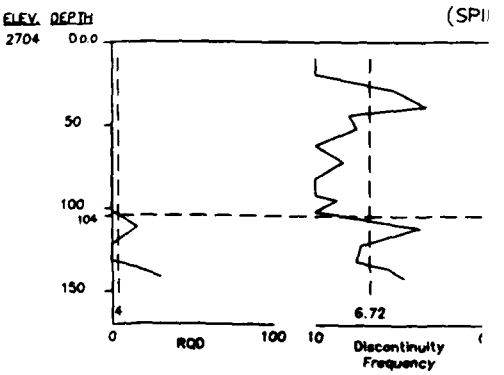
C86
(OUTLET WORKS,



C86
(GOVERNMENT

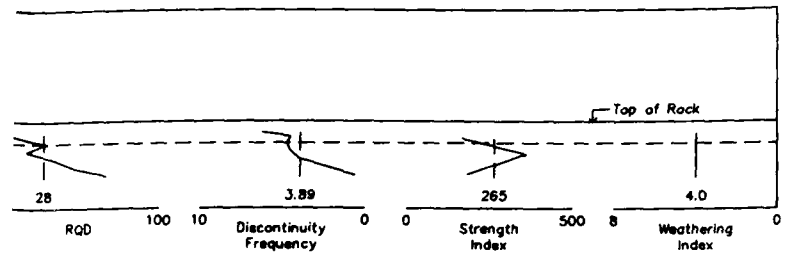


C8
(SPI

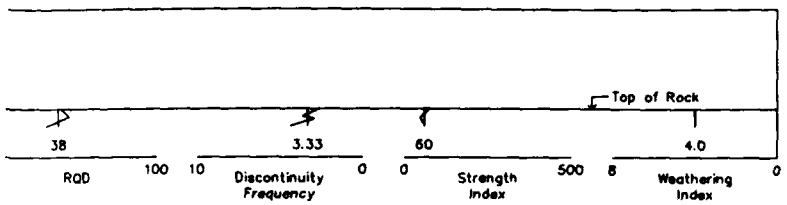


ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

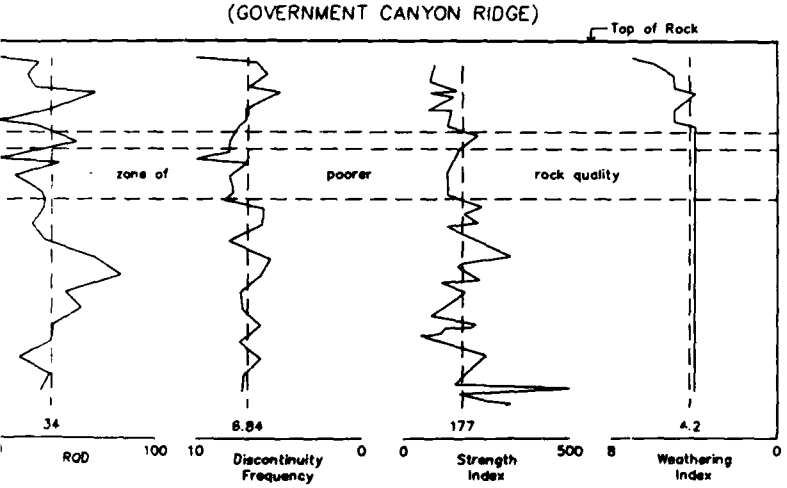
C86-12
(OUTLET WORKS, OUTLET PORTAL)



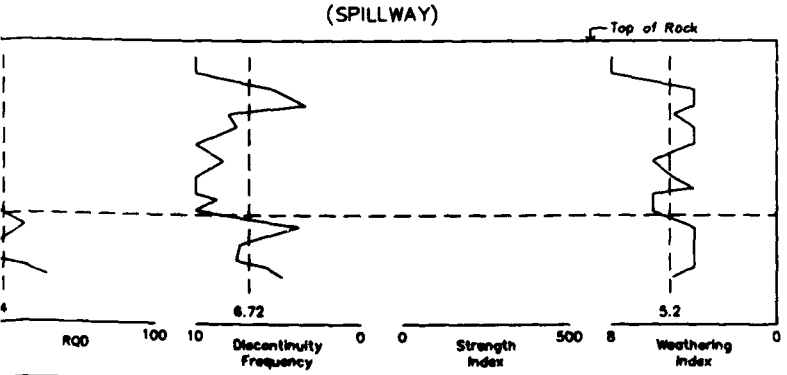
C86-13
(OUTLET WORKS, OUTLET PORTAL)



C86-14
(GOVERNMENT CANYON RIDGE)



C86-15
(SPILLWAY)



EXPLANATION

RQD — The total length of intact core pieces greater than 4 inches in length divided by the length of each core run. The RQD (Rock Quality Designation) is expressed as a percentage.

Discontinuity Frequency — The number of incipient or broken discontinuities (fractures, joints, shears, etc.) per foot of recovered core. A value of 10.00 is arbitrarily considered to be the maximum.

Strength Index — The results of point load tests with size corrected to represent 50mm core. As a rough approximation, the unconfined compressive strength is about 20 times greater than the point load strength index. Results are expressed in psi. Strength index values over 500 psi (very hard rock) are plotted as 500.

Weathering Index — An arbitrary numerical scale of core sample weathering based upon field inspection using Bieniawski's (1979) classification method.

Value	Weathering Description
0	Unweathered
1	Unweathered to Slightly Weathered
2	Slightly Weathered
3	Slightly to Moderately Weathered
4	Moderately Weathered or Slightly to Highly Weathered
5	Moderately to Highly Weathered
6	Highly Weathered or Moderately to Completely Weathered
7	Highly to Completely Weathered
8	Completely Weathered

NOTE:

See plate A-30 for general notes.

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

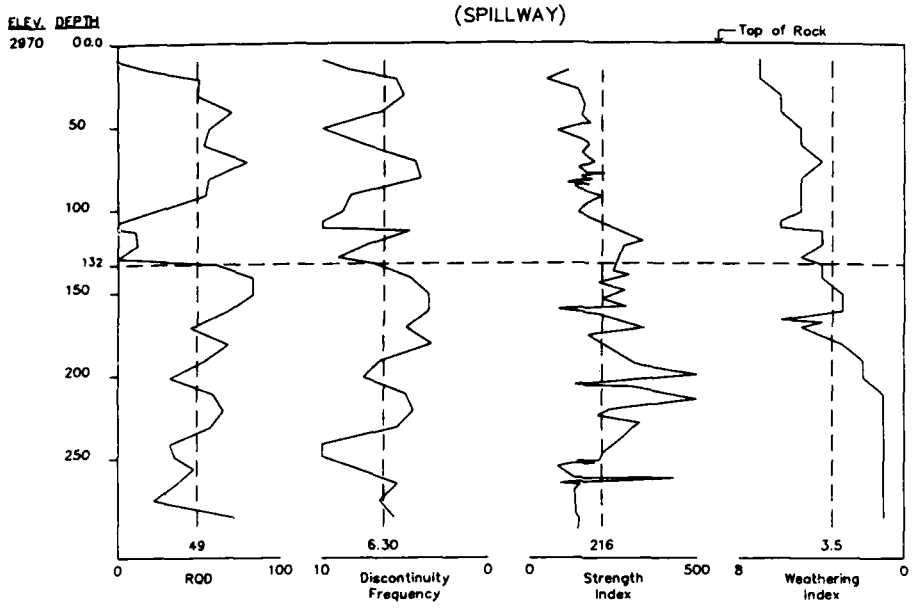
SEVEN OAKS DAM

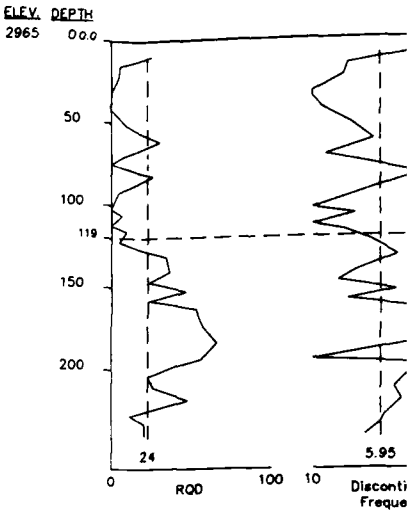
ROCK QUALITY PARAMETER COMPARISONS

SUBMITTED BY:
DATE APPROVED:
SPEC. NO. DACW09-____-____
DISTRICT FILE NO.
SHEET 3 OF 6 SHEETS

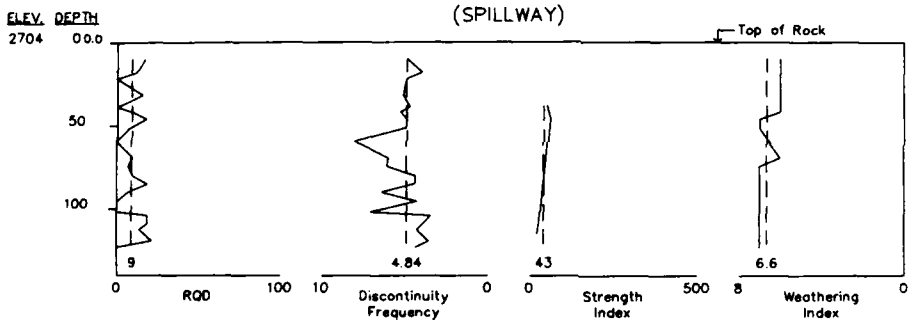
PLATE A-32

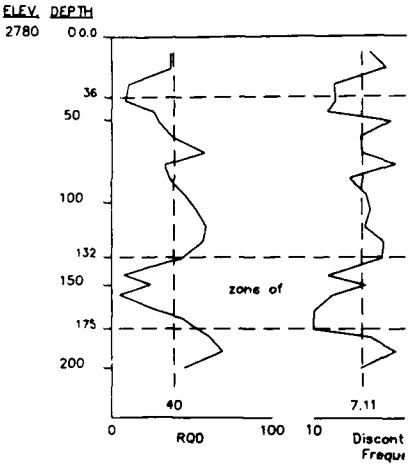
C86-16
(SPILLWAY)



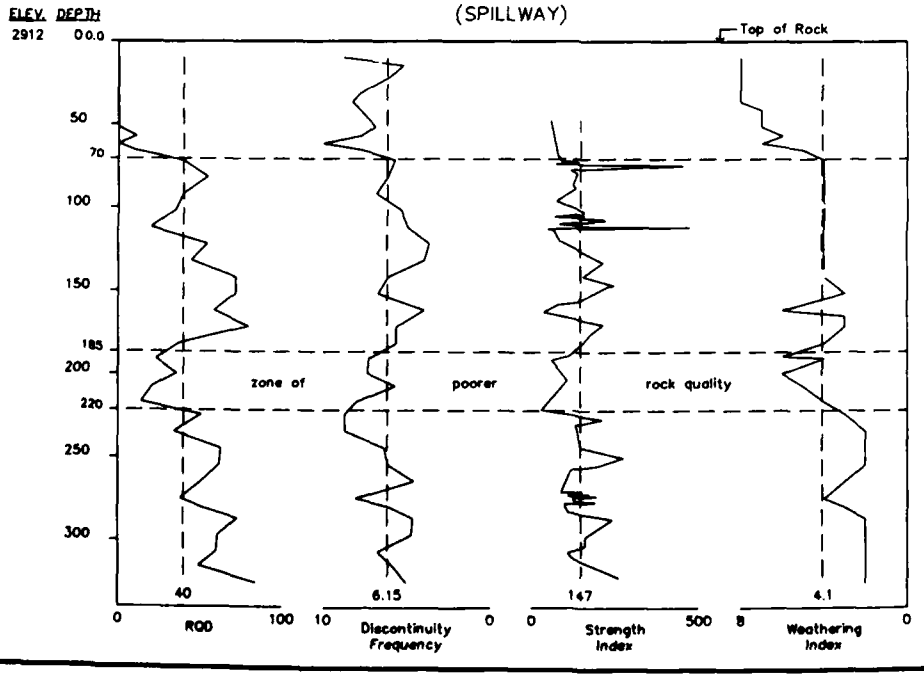


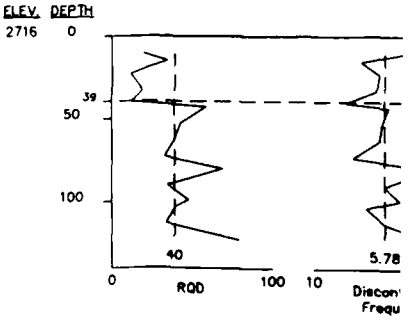
C86-17
(SPILLWAY)



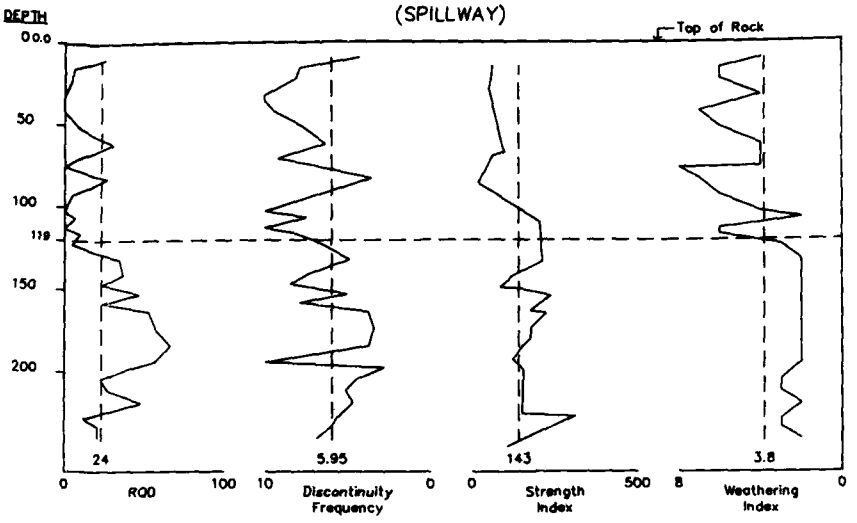


C86-18
(SPILLWAY)

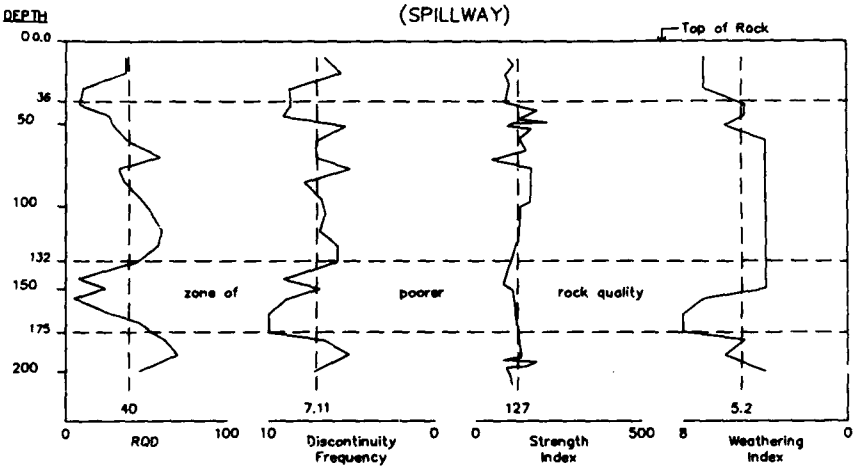




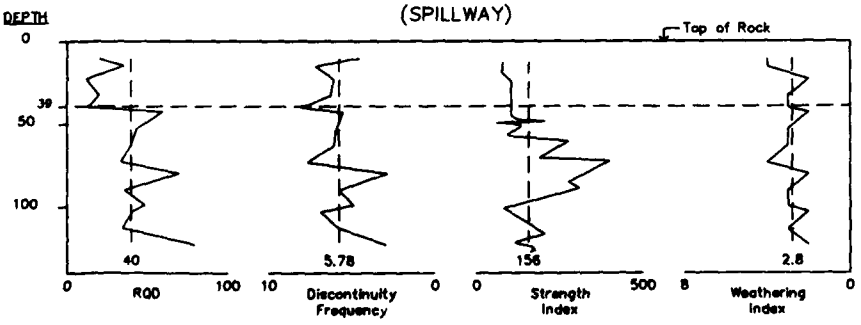
C86-19
(SPILLWAY)



C86-20
(SPILLWAY)



C86-21
(SPILLWAY)



EXPLANATION

RQD — The total length of intact core pieces greater than 4 inches in length divided by the length of each core run. The RQD (Rock Quality Designation) is expressed as a percentage.

Discontinuity Frequency — The number of incipient or broken discontinuities (fractures, joints, shears, etc.) per foot of recovered core. A value of 10.00 is arbitrarily considered to be the maximum.

Strength Index — The results of point load tests with size corrected to represent 50mm core. As a rough approximation, the unconfined compressive strength is about 20 times greater than the point load strength index. Results are expressed in psi. Strength index values over 500 psi (very hard rock) are plotted as 500.

Weathering Index — An arbitrary numerical scale of core sample weathering based upon field inspection using Bieniawski's (1979) classification method.

Value	Weathering Description
0	Unweathered
1	Unweathered to Slightly Weathered
2	Slightly Weathered
3	Slightly to Moderately Weathered
4	Moderately Weathered or Slightly to Highly Weathered
5	Moderately to Highly Weathered
6	Highly Weathered or Moderately to Completely Weathered
7	Highly to Completely Weathered
8	Completely Weathered

NOTE:

See plate A-30 for general notes.

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

ROCK QUALITY PARAMETER COMPARISONS

SUBMITTED BY: DATE APPROVED:

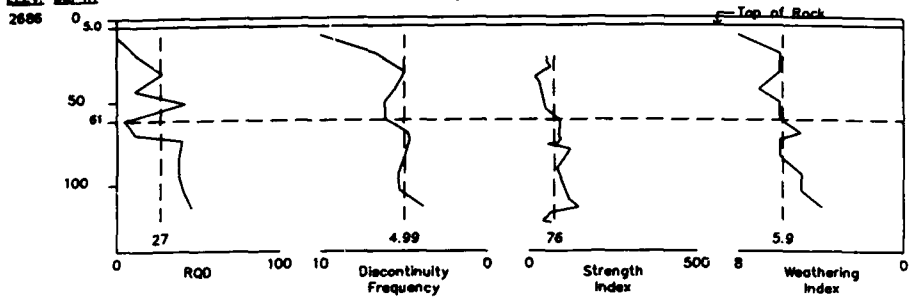
SPEC. NO. DACW09-__-__-__

DISTRICT FILE NO.

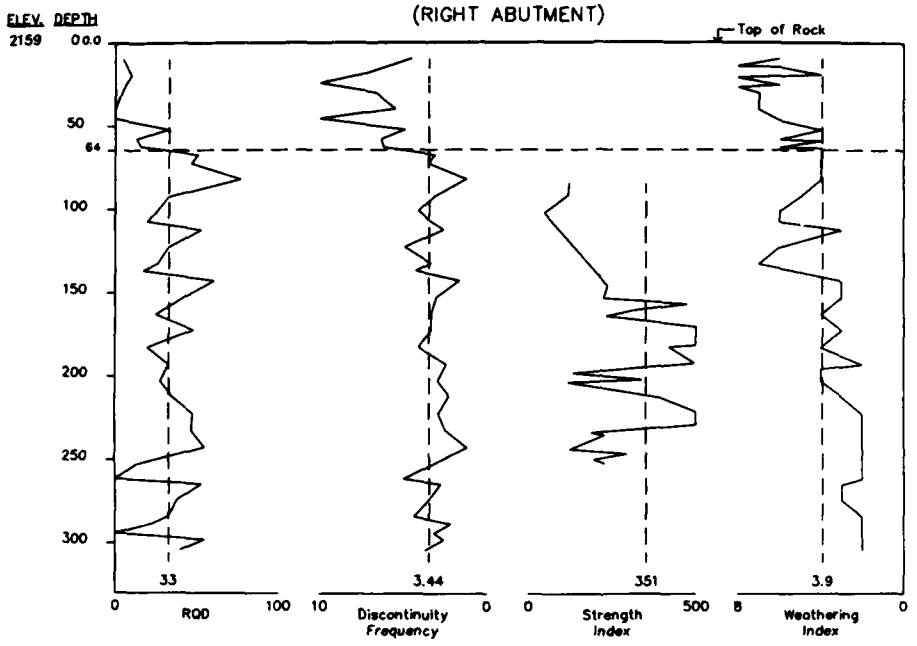
SHEET 4 OF 6 SHEETS

PLATE A-33

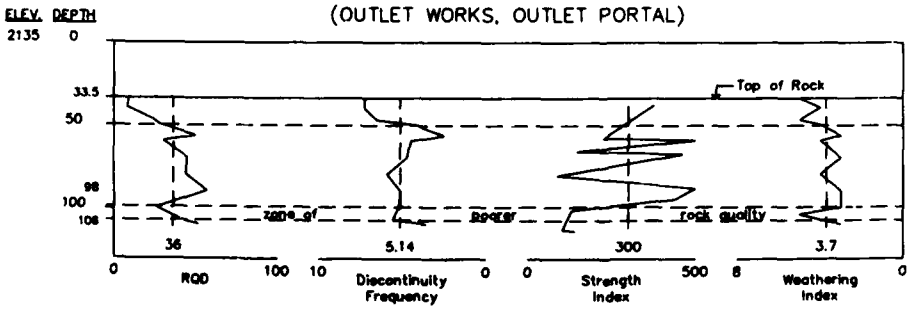
C86-22
(SPILLWAY)



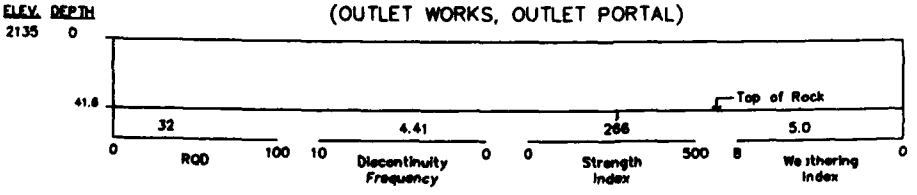
C86-23
(RIGHT ABUTMENT)



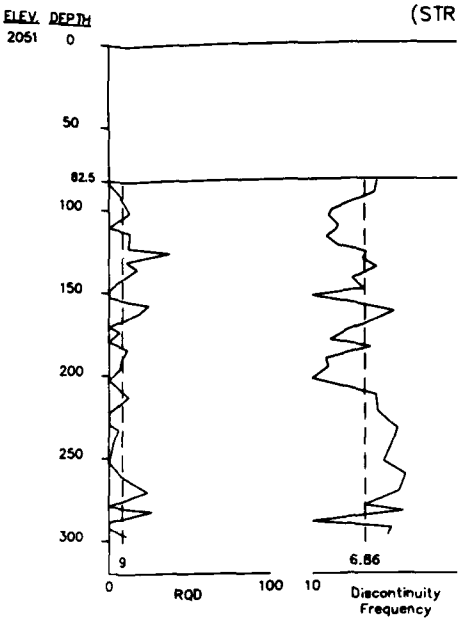
C86-24
(OUTLET WORKS, OUTLET PORTAL)



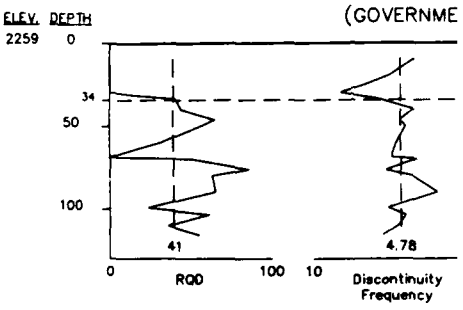
C86-25
(OUTLET WORKS, OUTLET PORTAL)



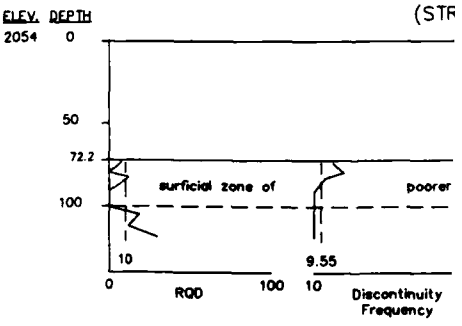
C
(STR



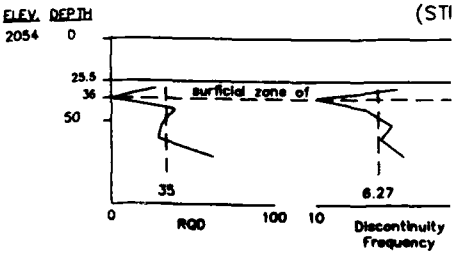
C
(GOVERNME



(STR



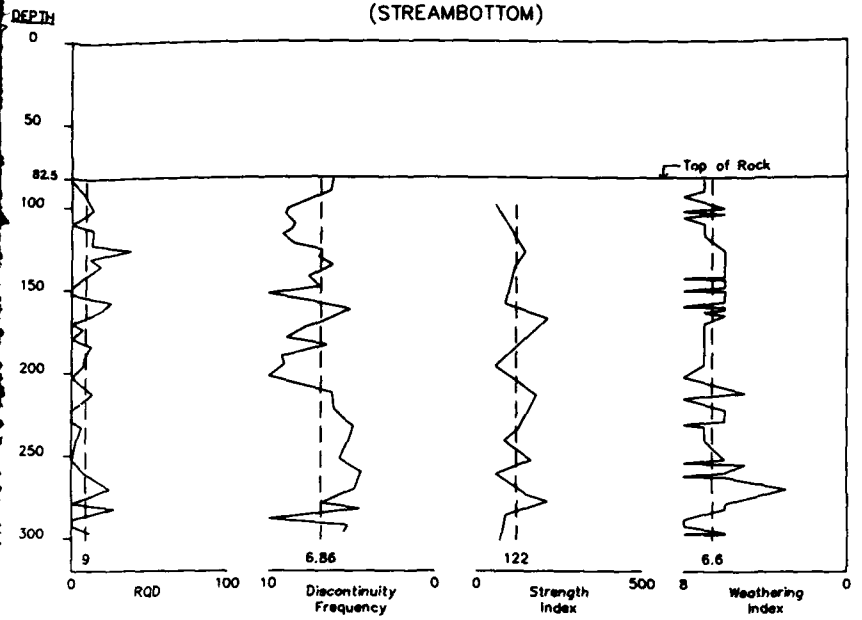
(STI



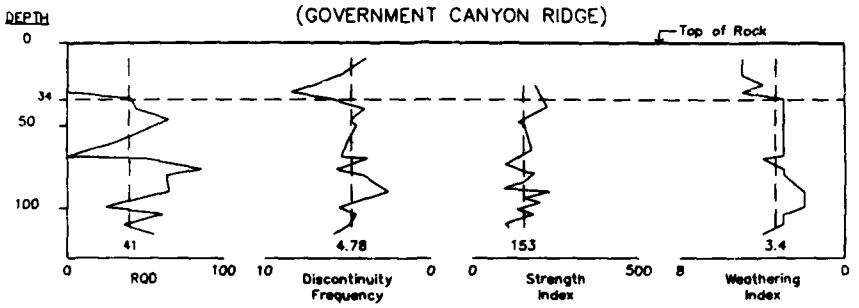
ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

VALUE ENGINEERING PAYS

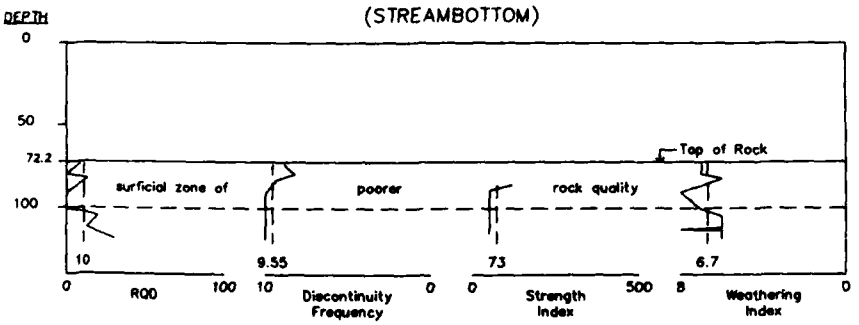
C86-26
(STREAMBOTTOM)



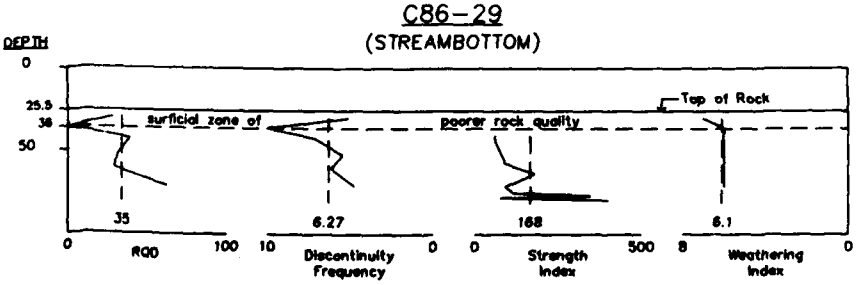
C86-27
(GOVERNMENT CANYON RIDGE)



C86-28
(STREAMBOTTOM)



C86-29
(STREAMBOTTOM)



EXPLANATION

RQD — The total length of intact core pieces greater than 4 inches in length divided by the length of each core run. The RQD (Rock Quality Designation) is expressed as a percentage.

Discontinuity Frequency — The number of incipient or broken discontinuities (fractures, joints, shears, etc.) per foot of recovered core. A value of 10.00 is arbitrarily considered to be the maximum.

Strength Index — The results of point load tests with size corrected to represent 50mm core. As a rough approximation, the unconfined compressive strength is about 20 times greater than the point load strength index. Results are expressed in psi. Strength index values over 500 psi (very hard rock) are plotted as 500.

Weathering Index — An arbitrary numerical scale of core sample weathering based upon field inspection using Bieniawski's (1979) classification method.

Value	Weathering Description
0	Unweathered
1	Unweathered to Slightly Weathered
2	Slightly Weathered
3	Slightly to Moderately Weathered
4	Moderately Weathered or Slightly to Highly Weathered
5	Moderately to Highly Weathered
6	Highly Weathered or Moderately to Completely Weathered
7	Highly to Completely Weathered
8	Completely Weathered

NOTE:

See plate A-30 for general notes.

SYMBOL	DESCRIPTION	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

ROCK QUALITY PARAMETER COMPARISONS

SUBMITTED BY: DATE APPROVED:
CHIEF, SURVEY

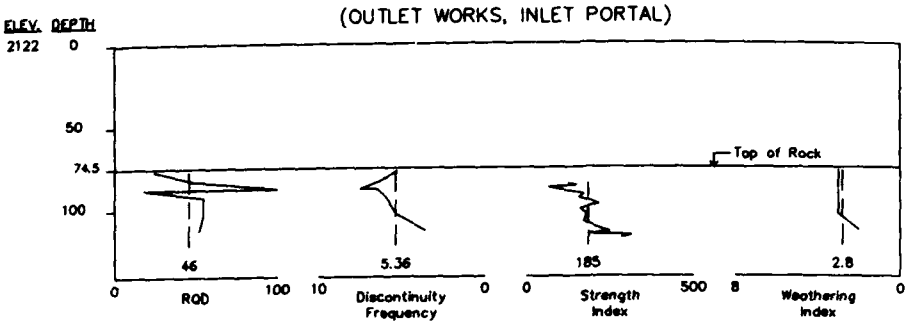
SPEC. NO. DACW09- -B- ----
DISTRICT FILE NO.

SHEET 5 OF 6 SHEETS

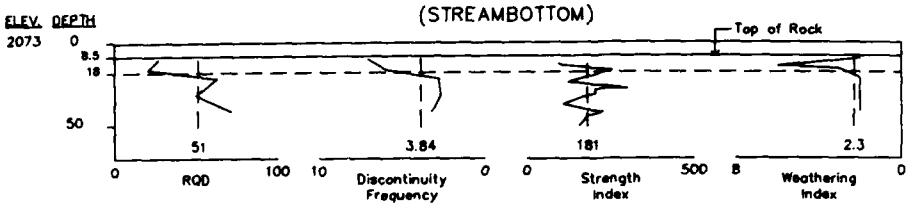
PLATE A-34

SAFETY PAYS

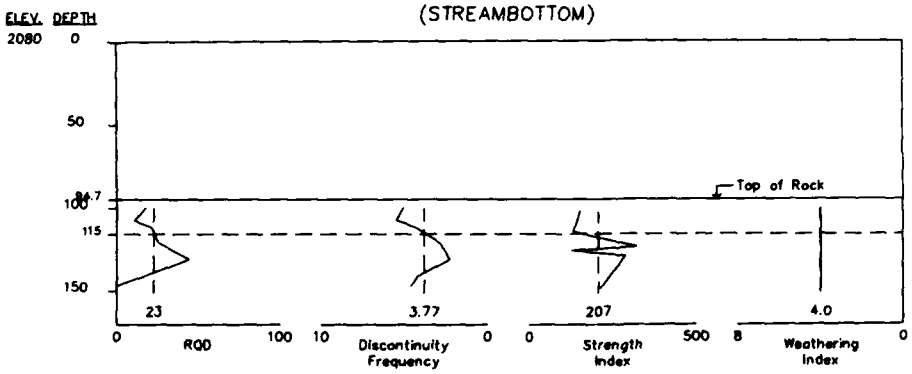
C86-30
(OUTLET WORKS, INLET PORTAL)



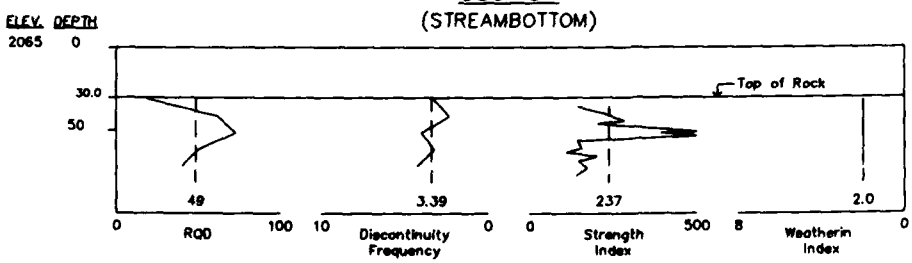
C86-32
(STREAMBOTTOM)



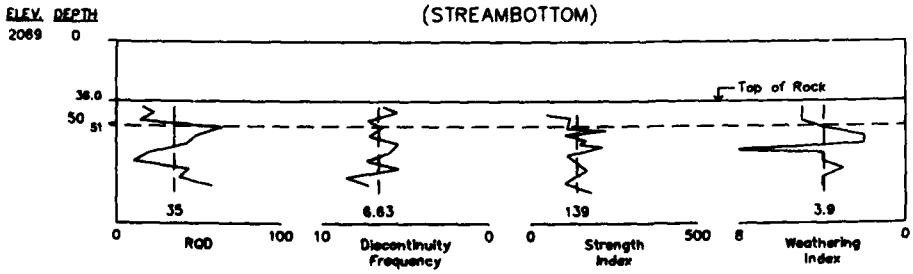
C86-33
(STREAMBOTTOM)



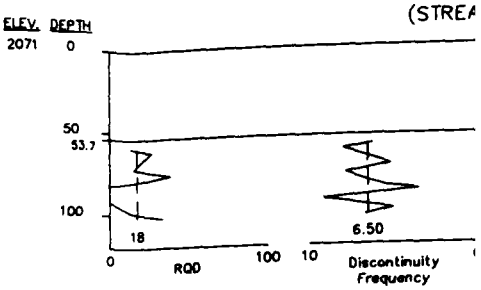
C86-34
(STREAMBOTTOM)



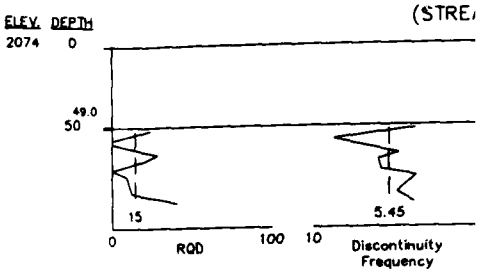
C86-35
(STREAMBOTTOM)



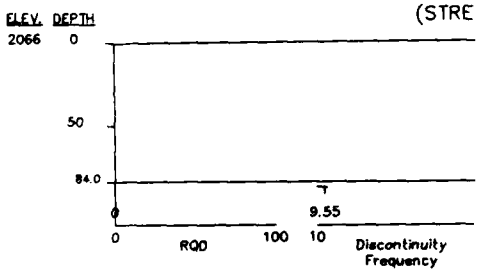
C8
(STREA



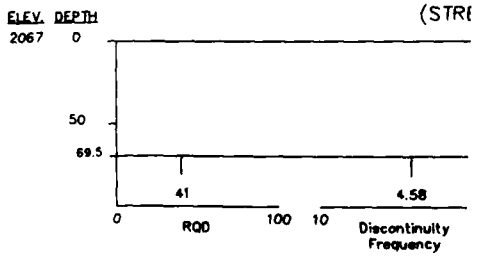
C8
(STREA



C8
(STRE



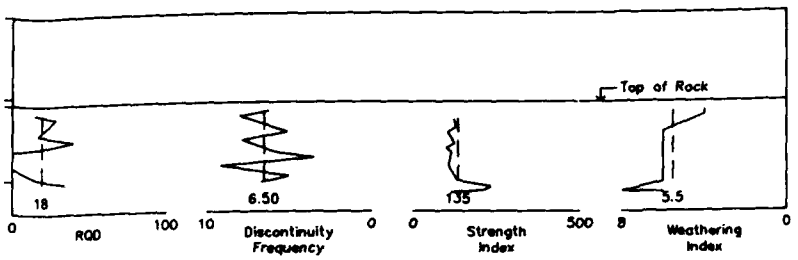
C
(STRE



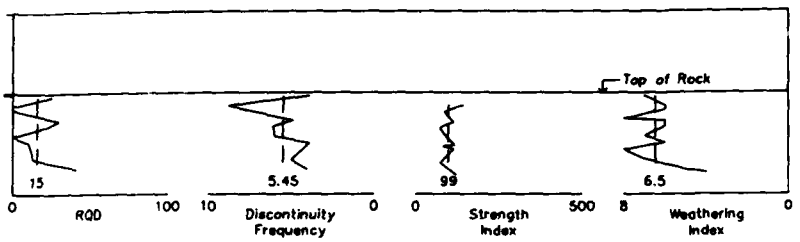
ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

VALUE ENGINEERING PAYS

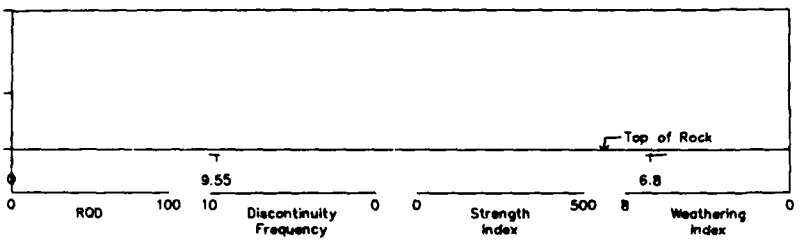
C86-36
(STREAMBOTTOM)



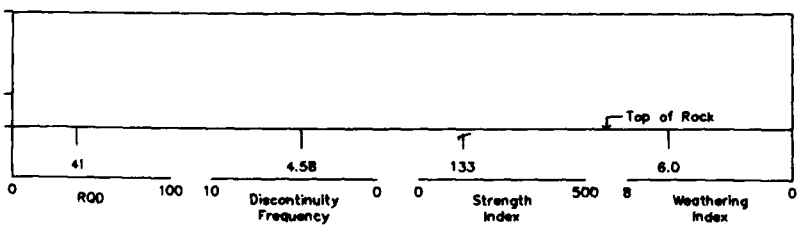
C86-37
(STREAMBOTTOM)



C86-38
(STREAMBOTTOM)



C86-39
(STREAMBOTTOM)



EXPLANATION

RQD — The total length of intact core pieces greater than 4 inches in length divided by the length of each core run. The RQD (Rock Quality Designation) is expressed as a percentage.

Discontinuity Frequency — The number of incipient or broken discontinuities (fractures, joints, shears, etc.) per foot of recovered core. A value of 10.00 is arbitrarily considered to be the maximum.

Strength Index — The results of point load tests with size corrected to represent 50mm core. As a rough approximation, the unconfined compressive strength is about 20 times greater than the point load strength index. Results are expressed in psi. Strength index values over 500 psi (very hard rock) are plotted as 500.

Weathering Index — An arbitrary numerical scale of core sample weathering based upon field inspection using Bieniawski's (1979) classification method.

Value	Weathering Description
0	Unweathered
1	Unweathered to Slightly Weathered
2	Slightly Weathered
3	Slightly to Moderately Weathered
4	Moderately Weathered or Slightly to Highly Weathered
5	Moderately to Highly Weathered
6	Highly Weathered or Moderately to Completely Weathered
7	Highly to Completely Weathered
8	Completely Weathered

NOTE:

See plate A-30 for general notes.

SYMBOL	DESCRIPTION	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

ROCK QUALITY PARAMETER COMPARISONS

SUBMITTED BY: DATE APPROVED:

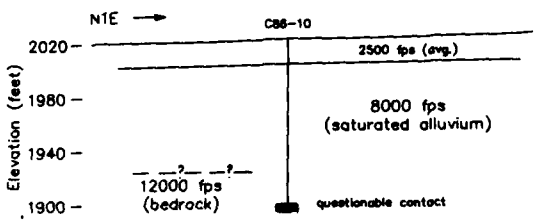
SPEC. NO. DACW09- ____ B- ____

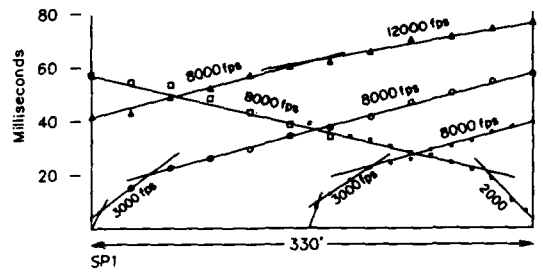
DISTRICT FILE NO.

SHEET 6 OF 6 SHEETS

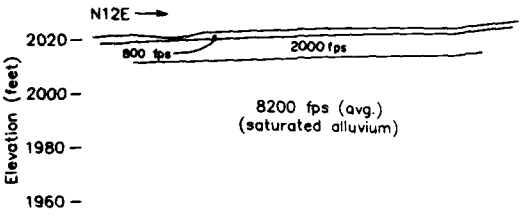
PLATE A-35

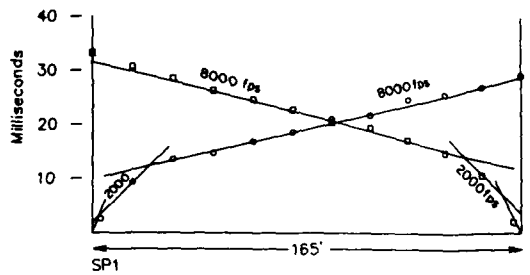
SAFETY PAYS



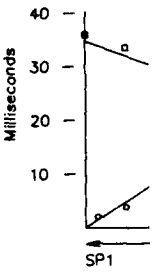


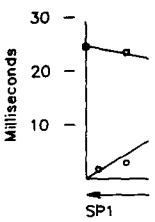
84-1/2



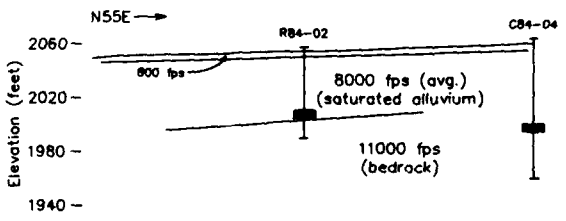


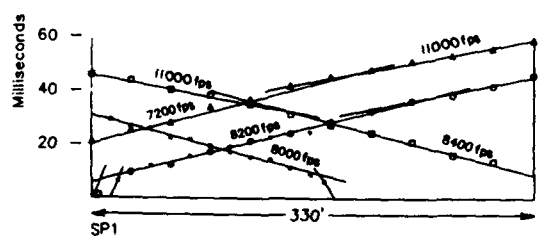
84-3



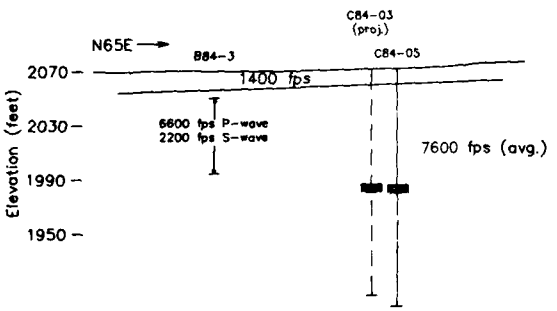


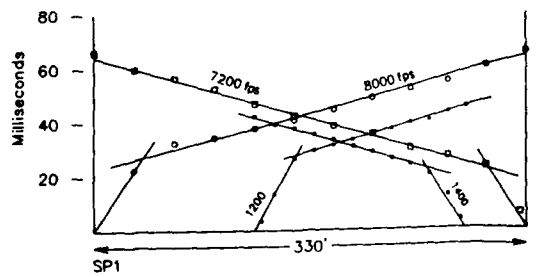
ENVIRONMENTAL
ENHANCEMENT
THRU ENGINEERING



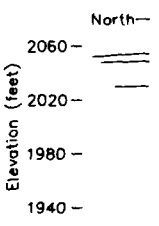


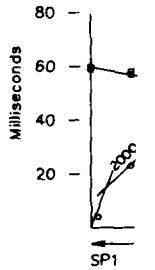
84-6/7



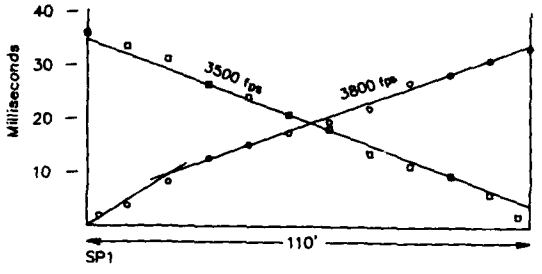


84-8/9



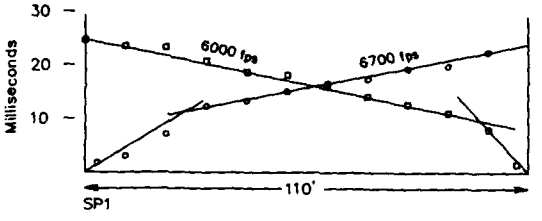


ALUE ENGINEERING PAYS



84–5

(gneiss outcrop)



84–4

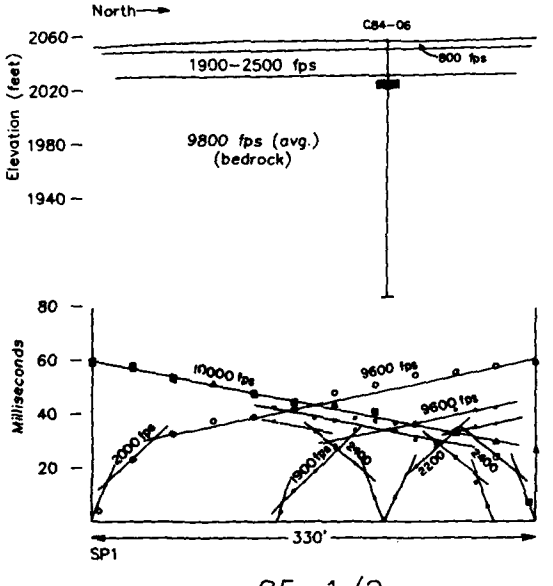
(diorite outcrop)

LEGEND:

- ○ Shot Point 1 (SP1)
- □ Shot Point 2 (SP2)
- △ Shot Point 3 (SP3)
- ◇ Shot Point 4 (SP4)
- ⊕ Shot Point 5 (SP5)

GENERAL NOTES:

1. See plate A–10 for locations of seismic refraction survey lines.
2. See attachment A–4 for results of seismic refraction surveys conducted by Leighton and Associates.
3. See plates A–11 to A–29 for drill logs of exploratory rotary and core holes.
4. See section 7 of this appendix for a discussion of the results and interpretation of seismic refraction surveys.
5. Interface depths were calculated using the critical distance method.



85–1/2

NOTES:

1. Where two lines are plotted together (i.e. 84–8/9), separate lines with different geophone spacings were conducted at the same location.
2. Seismic refraction lines 84–4 and 84–5 were conducted adjacent to diorite and gneiss outcrops in order to determine near surface bedrock velocities.

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

SEISMIC REFRACTION SURVEYS
TRAVELTIME CURVES AND INTERPRETATIONS

SUBMITTED BY: DATE APPROVED:
CHIEF, DIVISION

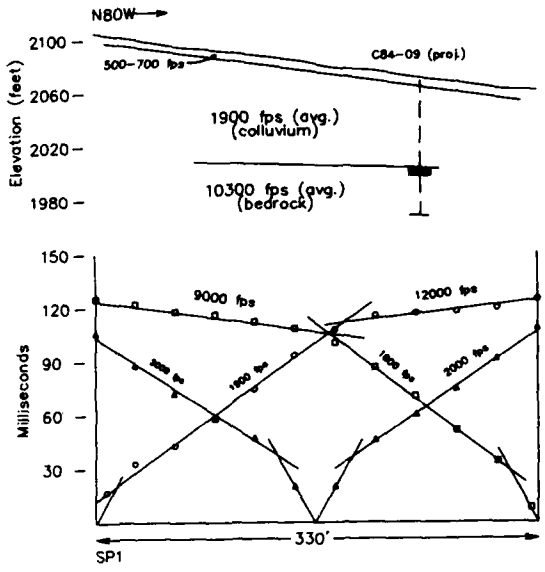
SPEC. NO. DACW09-___-___-___
DISTRICT FILE NO.

SHEET 1 OF 4 SHEETS

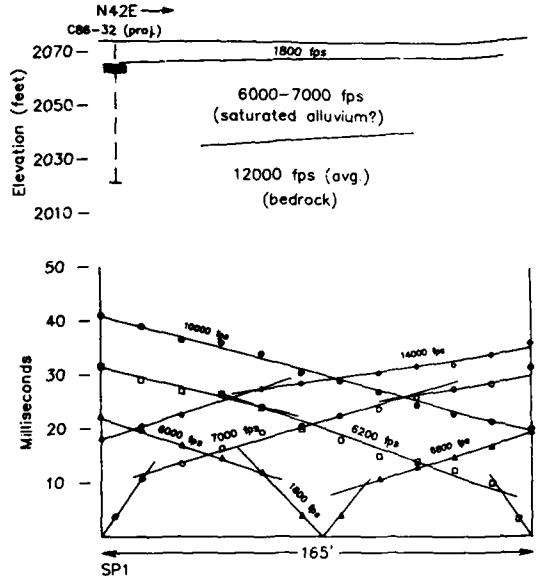
PLATE A-36

SAFETY PAYS

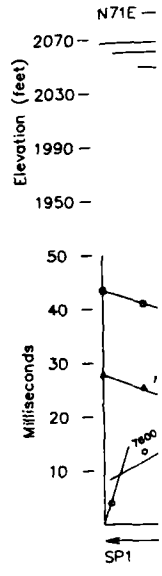
ENVIRONMENTAL
ENHANCEMENT
THRU ENGINEERING

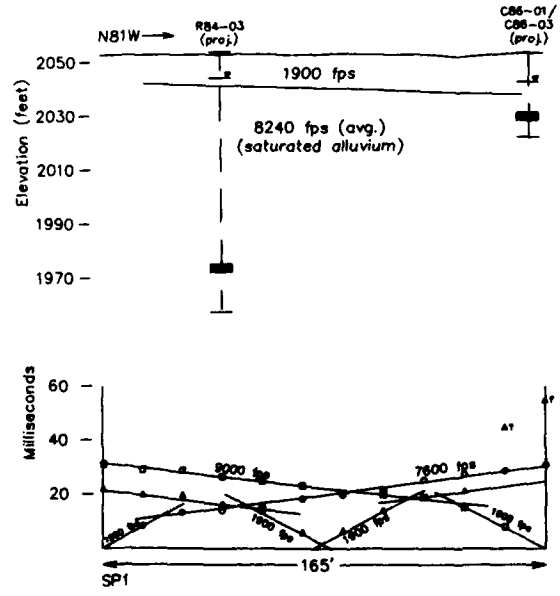


85-3

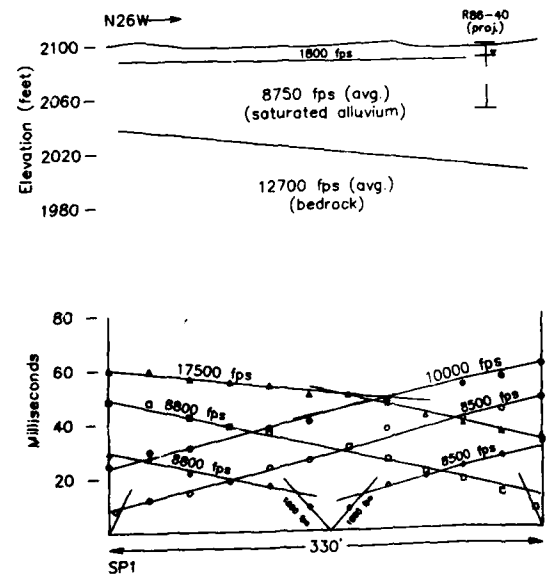


85-4

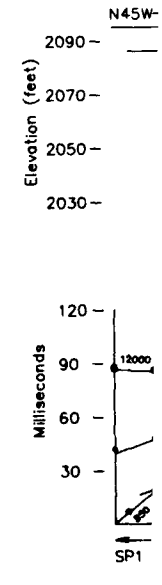




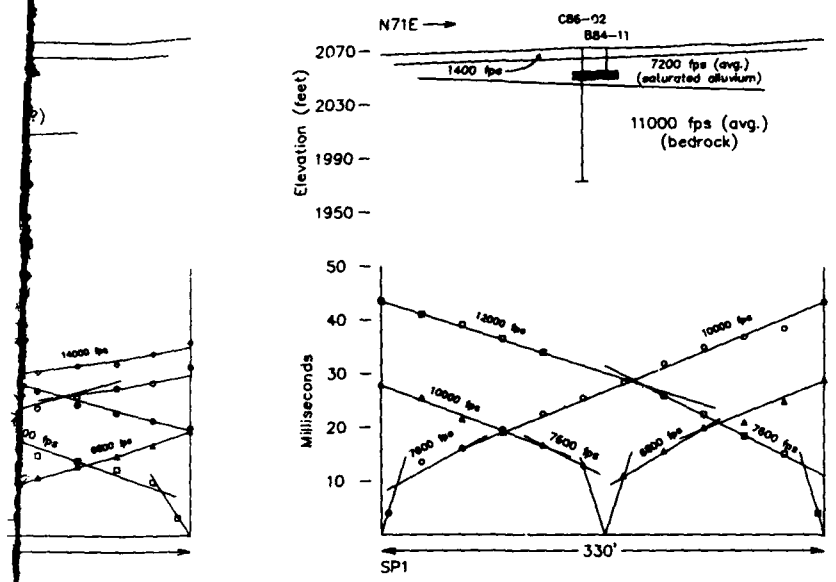
86-01



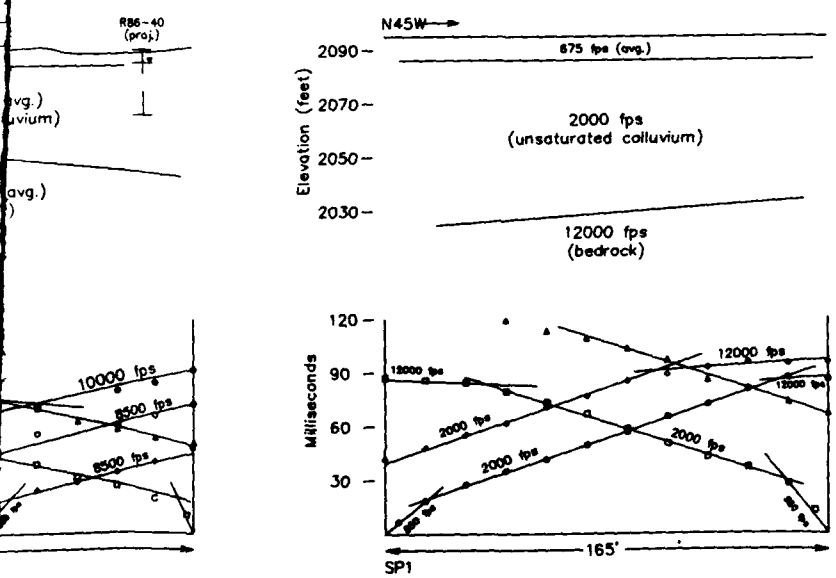
86-02



VALUE ENGINEERING PAYS



85-5



86-03

LEGEND:

- ○ Shot Point 1 (SP1)
- □ Shot Point 2 (SP2)
- △ Shot Point 3 (SP3)
- ◇ Shot Point 4 (SP4)
- ⊕ Shot Point 5 (SP5)

GENERAL NOTES:

1. See plate A-10 for locations of seismic refraction survey lines.
2. See attachment A-4 for results of seismic refraction surveys conducted by Leighton and Associates.
3. See plates A-11 to A-29 for drill logs of exploratory rotary and core holes.
4. See section 7 of this appendix for a discussion of the results and interpretation of seismic refraction surveys.
5. Interface depths were calculated using the critical distance method.

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
REVISIONS			

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SEVEN OAKS DAM

SEISMIC REFRACTION SURVEYS
TRAVELTIME CURVES AND INTERPRETATIONS

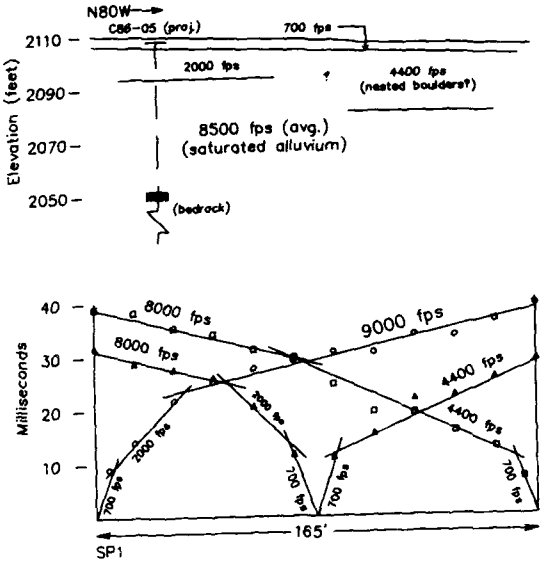
SUBMITTED BY: DATE APPROVED:
CHIEF, BRANCH

SPEC. NO. DACW09-____-B-____
DISTRICT FILE NO.

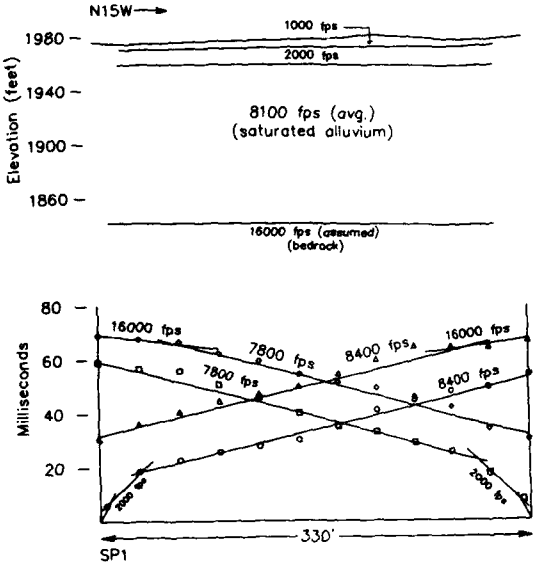
SHEET 2 OF 4 SHEETS

PLATE A-37

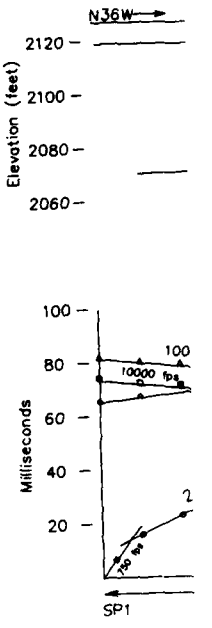
SAFETY PAYS



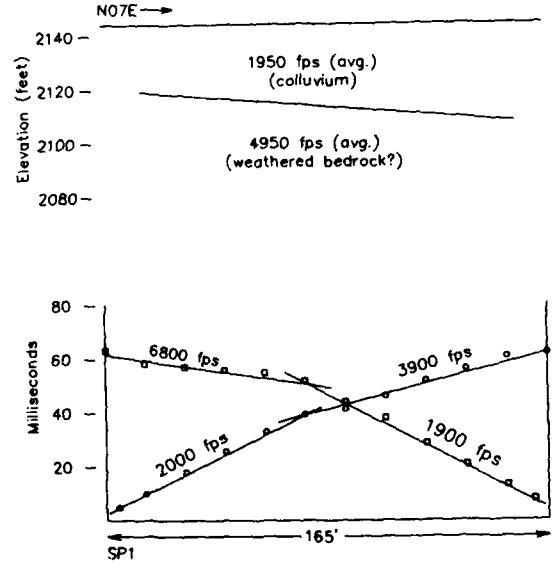
86-05



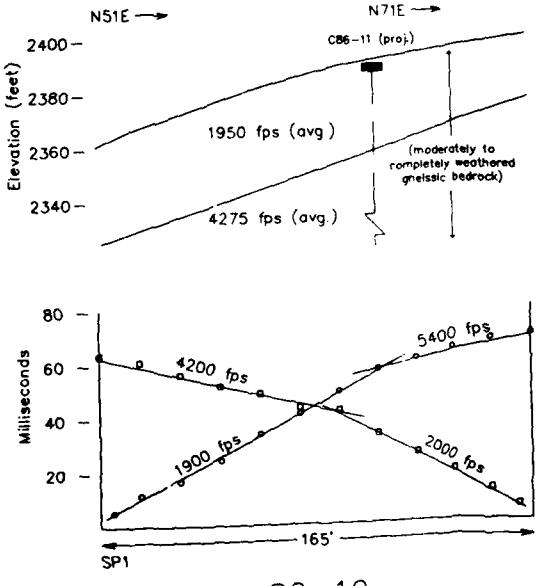
86-07



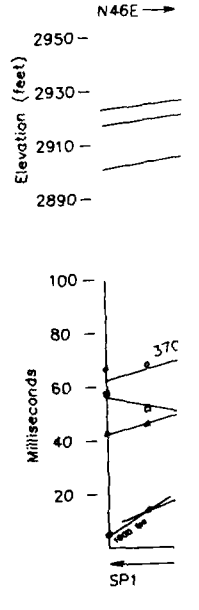
ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

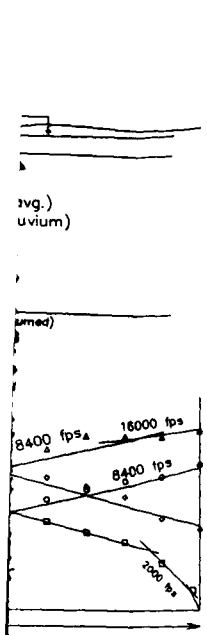


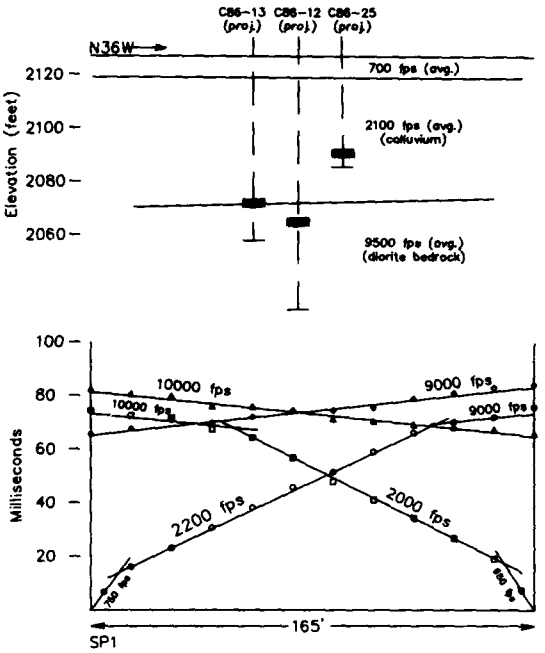
86-09



86-10







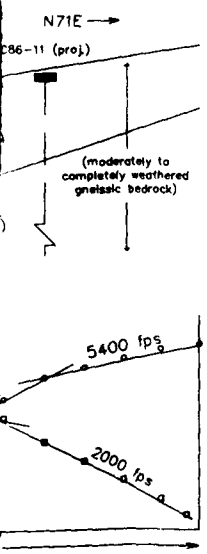
86-08

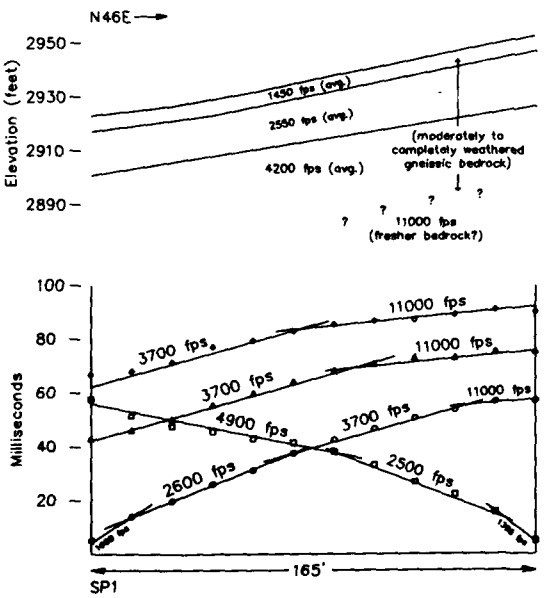
LEGEND:

- ○ Shot Point 1 (SP1)
- □ Shot Point 2 (SP2)
- △ Shot Point 3 (SP3)
- ◇ Shot Point 4 (SP4)
- ⊕ Shot Point 5 (SP5)

GENERAL NOTES:

1. See plate A-10 for locations of seismic refraction survey lines.
2. See attachment A-4 for results of seismic refraction surveys conducted by Leighton and Associates.
3. See plates A-11 to A-29 for drill logs of exploratory rotary and core holes.
4. See section 7 of this appendix for a discussion of the results and interpretation of seismic refraction surveys.
5. Interface depths were calculated using the critical distance method.





86-11

NOTE:

1. Seismic refraction lines 86-04 and 86-06 were conducted downstream at an alternative damsite. Results of these are not pertinent to the selected damsite and are not shown.

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
REVISIONS			

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

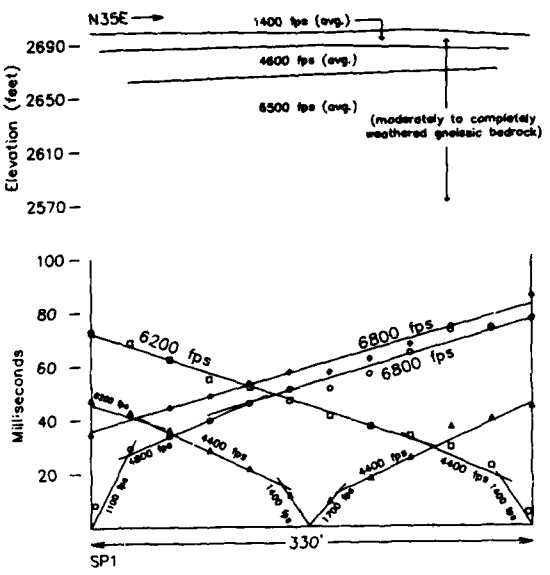
SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

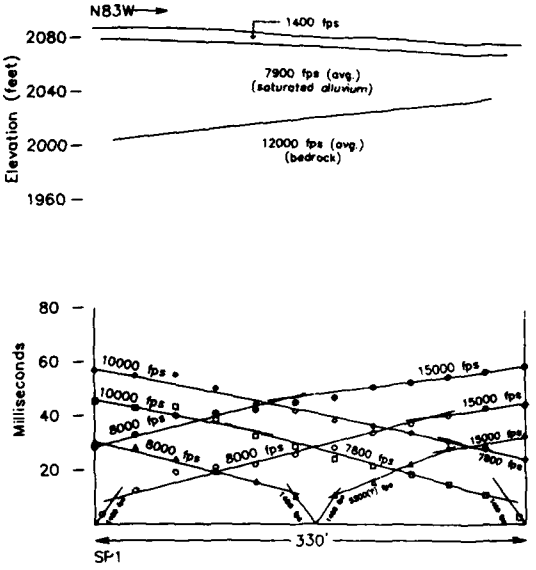
SEISMIC REFRACTION SURVEYS
TRAVELTIME CURVES AND INTERPRETATIONS

SUBMITTED BY:
DATE APPROVED:
SPEC. NO. DACW09-____-____
DISTRICT FILE NO.
SHEET 3 OF 4 SHEETS

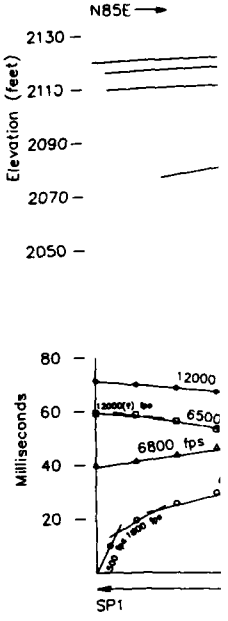
PLATE A-38

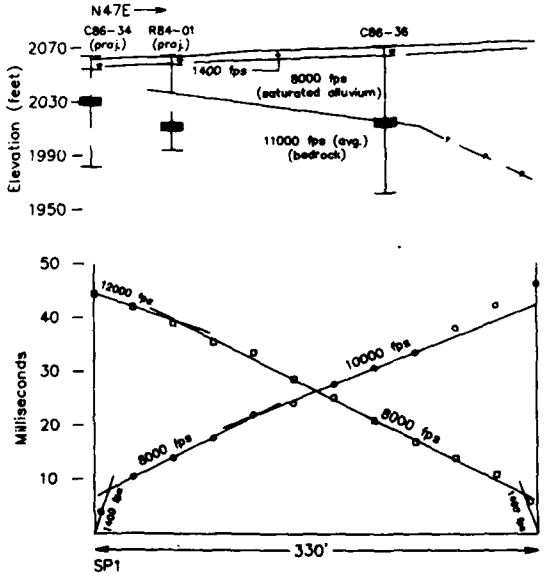


87-01

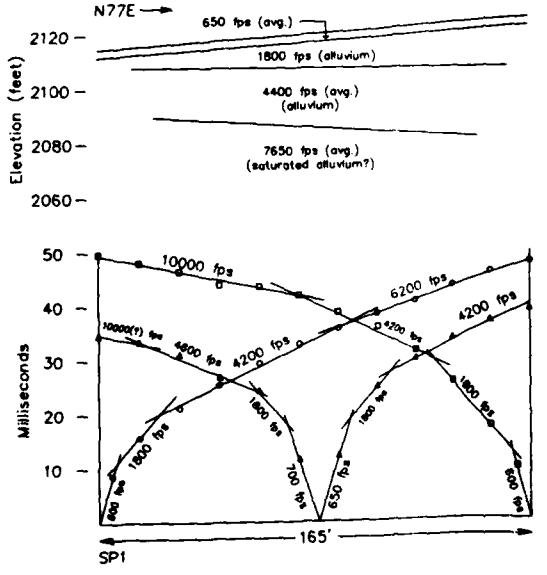


87-02

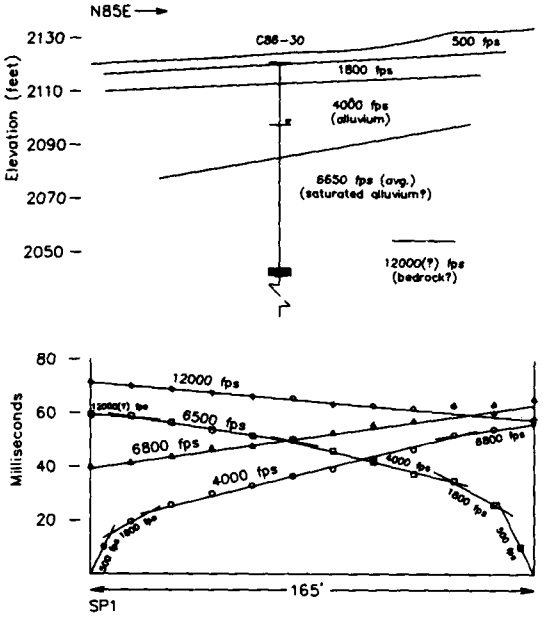




87-04



87-05



87-03

LEGEND:

- ○ Shot Point 1 (SP1)
- □ Shot Point 2 (SP2)
- △ Shot Point 3 (SP3)
- ◇ Shot Point 4 (SP4)
- ⊕ Shot Point 5 (SP5)

GENERAL NOTES:

1. See plate A-10 for locations of seismic refraction survey lines.
2. See attachment A-4 for results of seismic refraction surveys conducted by Leighton and Associates.
3. See plates A-11 to A-29 for drill logs of exploratory rotary and core holes.
4. See section 7 of this appendix for a discussion of the results and interpretation of seismic refraction surveys.
5. Interface depths were calculated using the critical distance method.

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

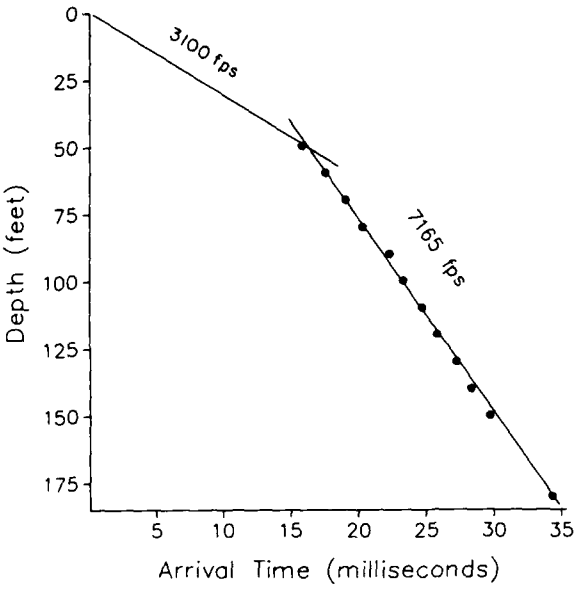
SEISMIC REFRACTION SURVEYS
TRAVELTIME CURVES AND INTERPRETATIONS

SUBMITTED BY: DATE APPROVED:
CHIEF, ENGINEERING DIVISION

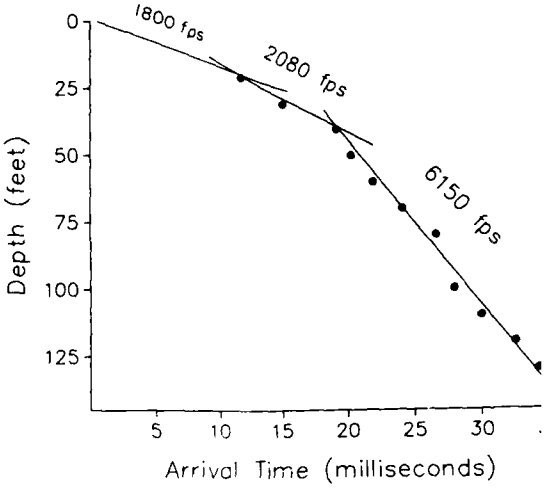
SPEC. NO. DACW09-__-__-__
DISTRICT FILE NO.

SHEET 4 OF 4 SHEETS

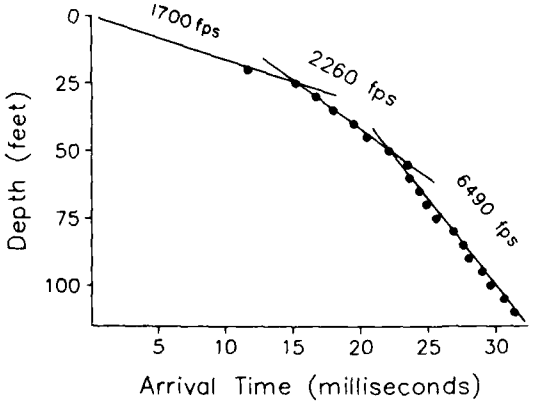
PLATE A-39



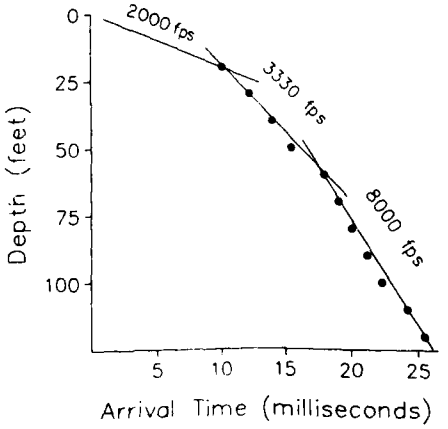
C86-14



C86-19

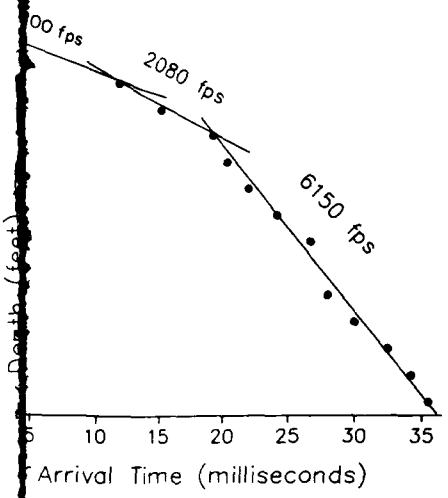


C86-22

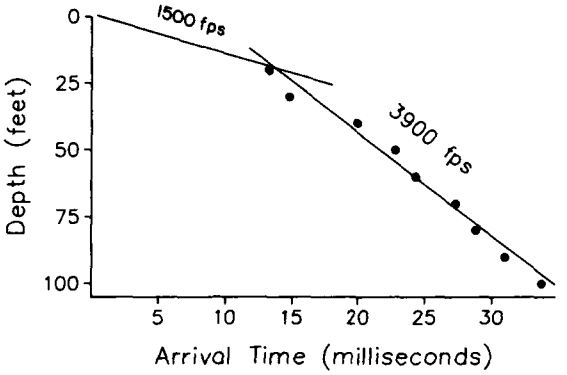


C86-27

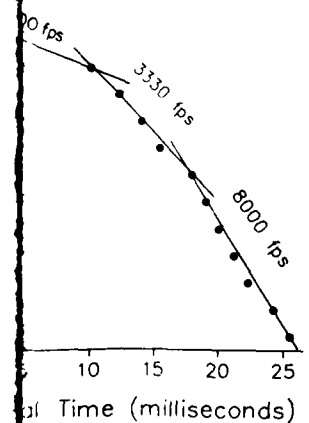
ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING



C86-19



C86-20



C86-27

NOTES:

1. Downhole seismic surveys were conducted in selected holes in the spillway and Government Canyon Ridge. See plate A-9 for hole locations.
2. See plates A-25, A-28 and A-29 for geologic drill logs.

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

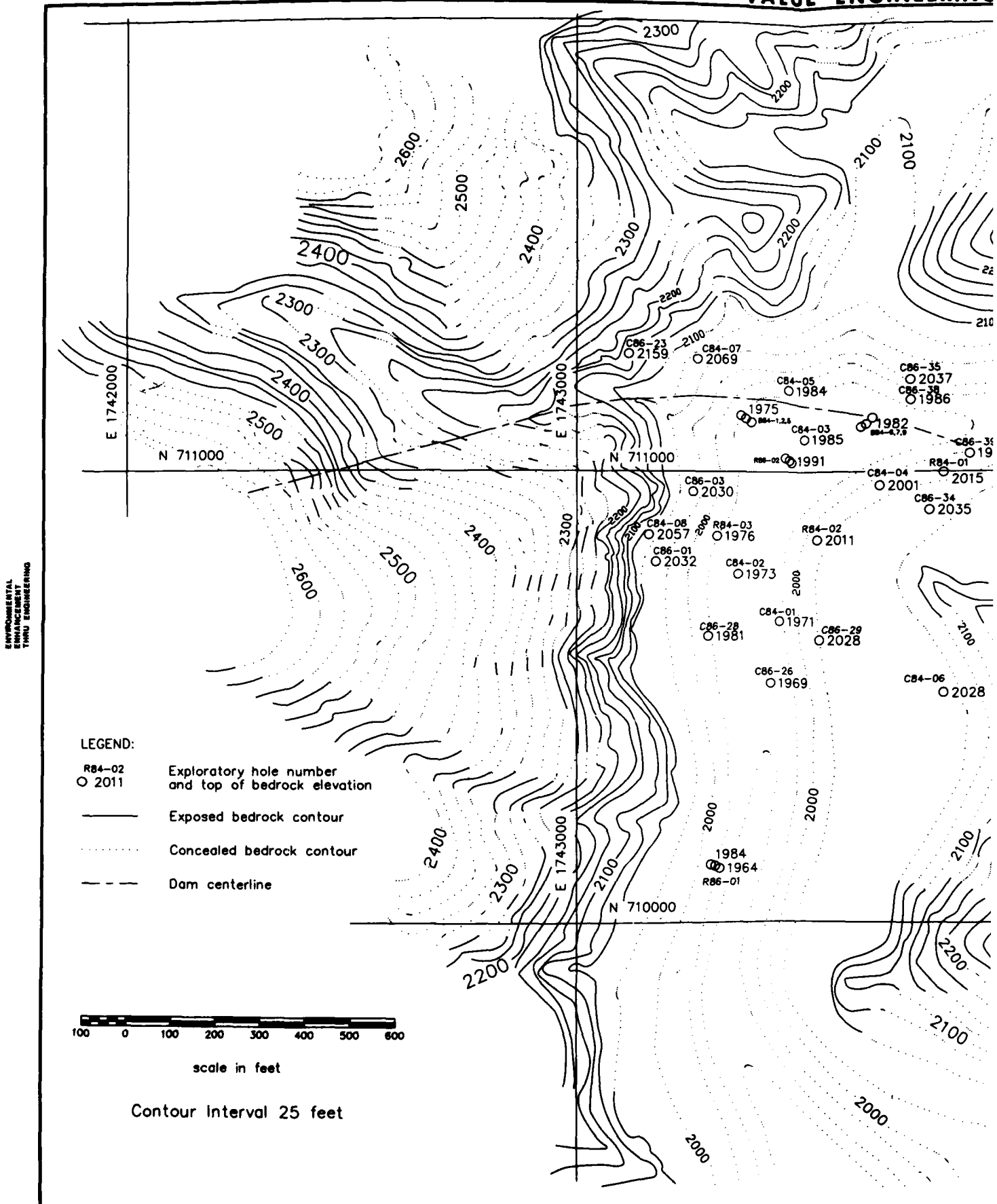
DOWNHOLE SEISMIC GRAPHS

SUBMITTED BY: DATE APPROVED:

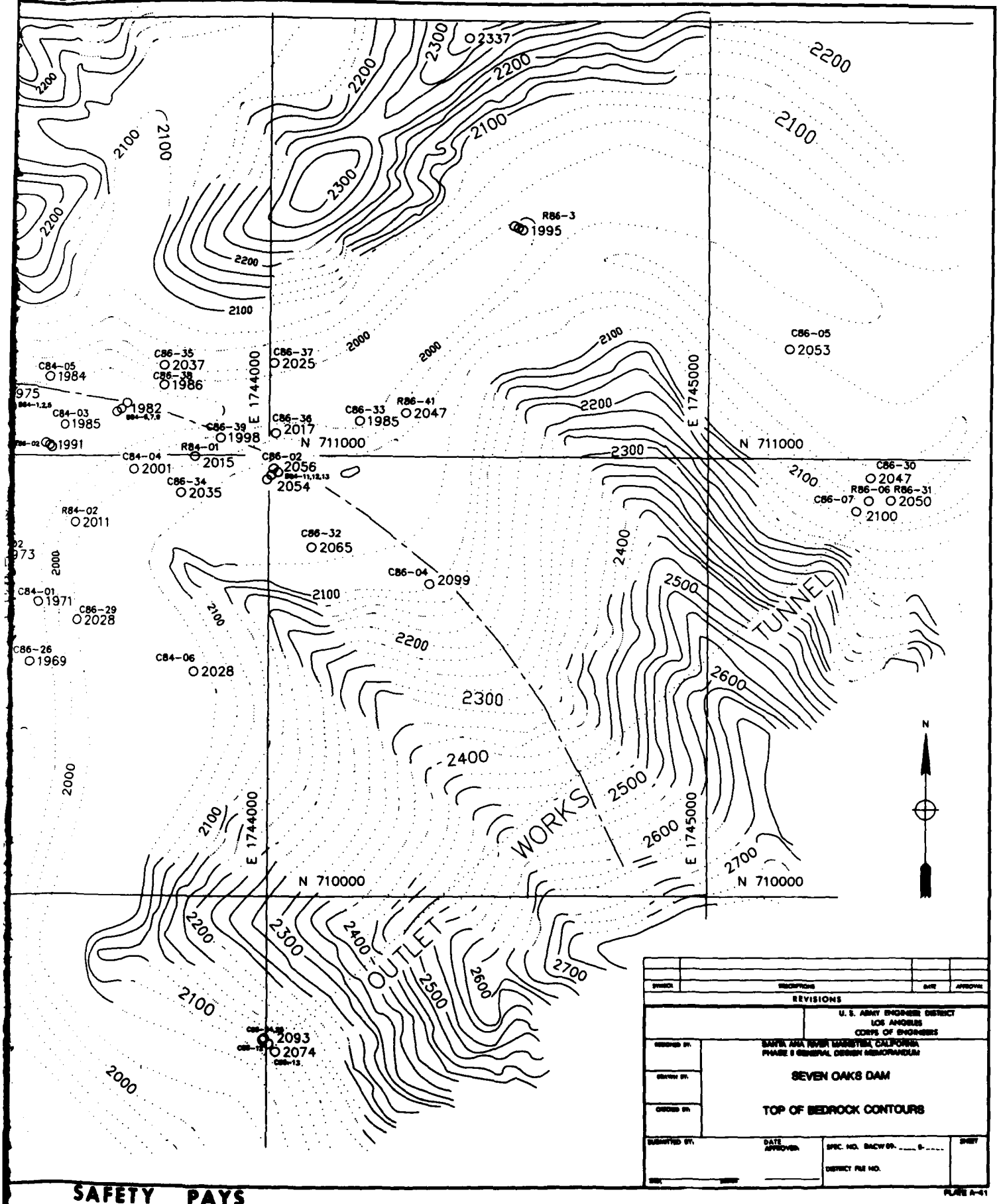
SPEC. NO. DACW09- -B- SHEET

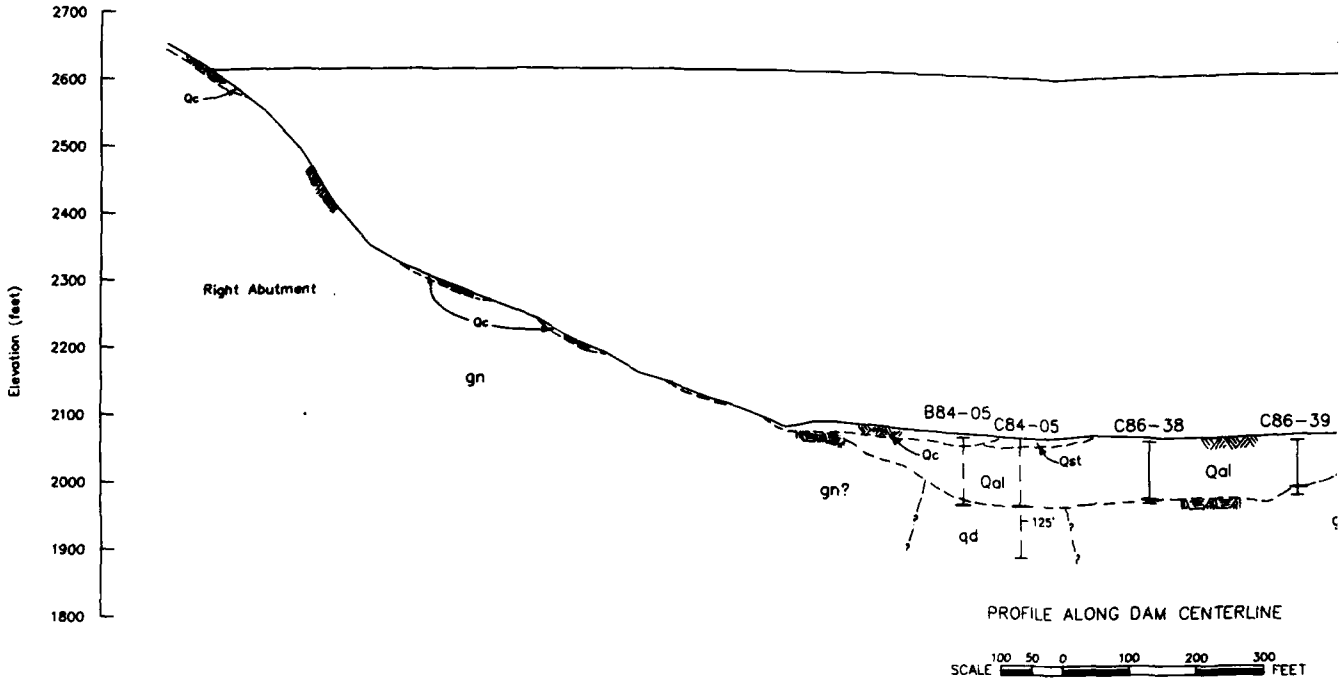
DISTRICT FILE NO.

PLATE A-40



ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

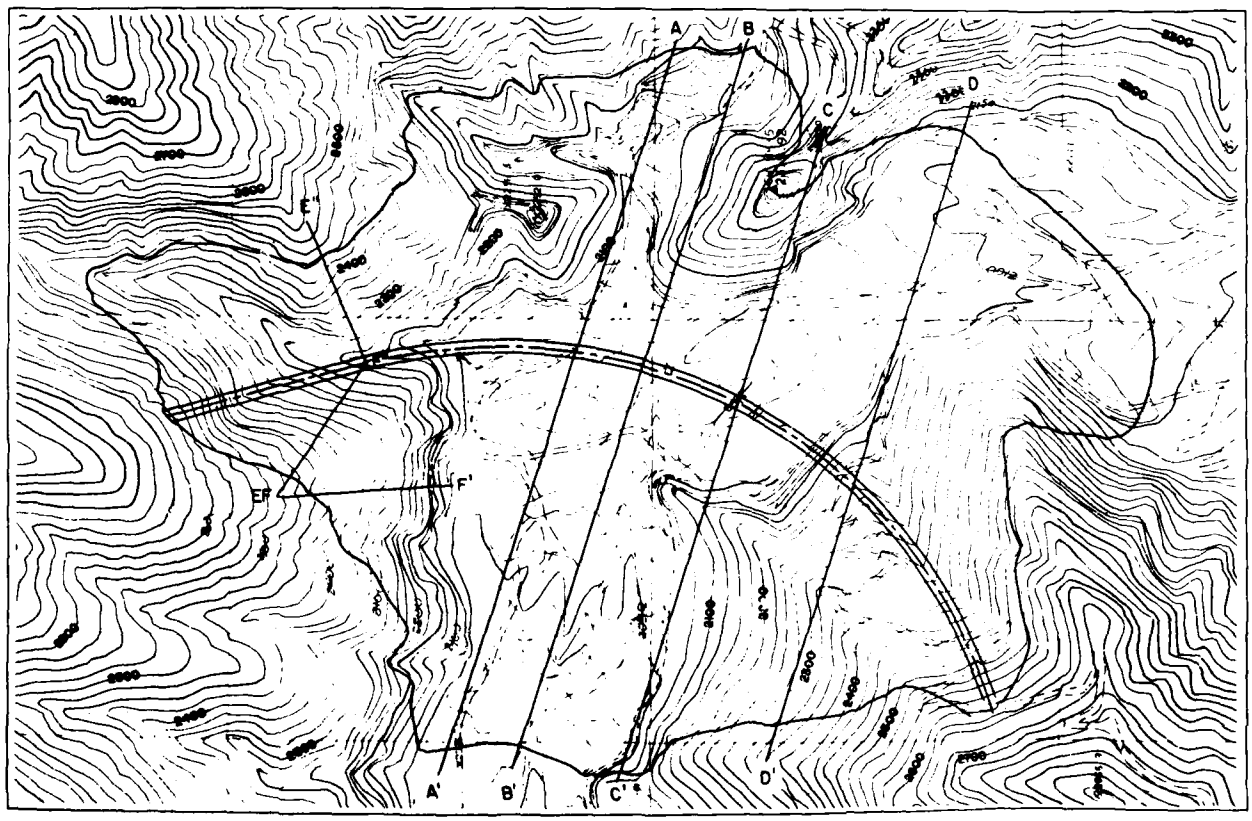

2200
2300
2337
2100
2000
R86-3
1995
C86-05
2053
C84-05
1984
C86-35
2037
C86-38
1986
C86-37
2025
E 1744000
E 1745000
C84-03
1985
1982
C86-39
1998
C86-36
2017
C86-33
1985
R86-41
2047
1991
R84-01
2015
C84-04
2001
C86-02
2056
2054
N 711000
C86-34
2035
C86-30
2047
R86-06 R86-31
C86-07
2050
2100
R84-02
2011
C86-32
2065
C86-04
2099
2400
2500
TUNNEL
C84-01
1971
C86-29
2028
C86-26
1969
C84-06
2028
2600
WORKS
2700
N 710000
OUTLET
2093
2074
N
REVISIONS
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS
SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM
SEVEN OAKS DAM
TOP OF BEDROCK CONTOURS
DATE APPROVED
DISTRICT FILE NO.
PLATE A-41



PROFILE ALONG DAM CENTERLINE

SCALE 100 50 0 100 200 300 FEET

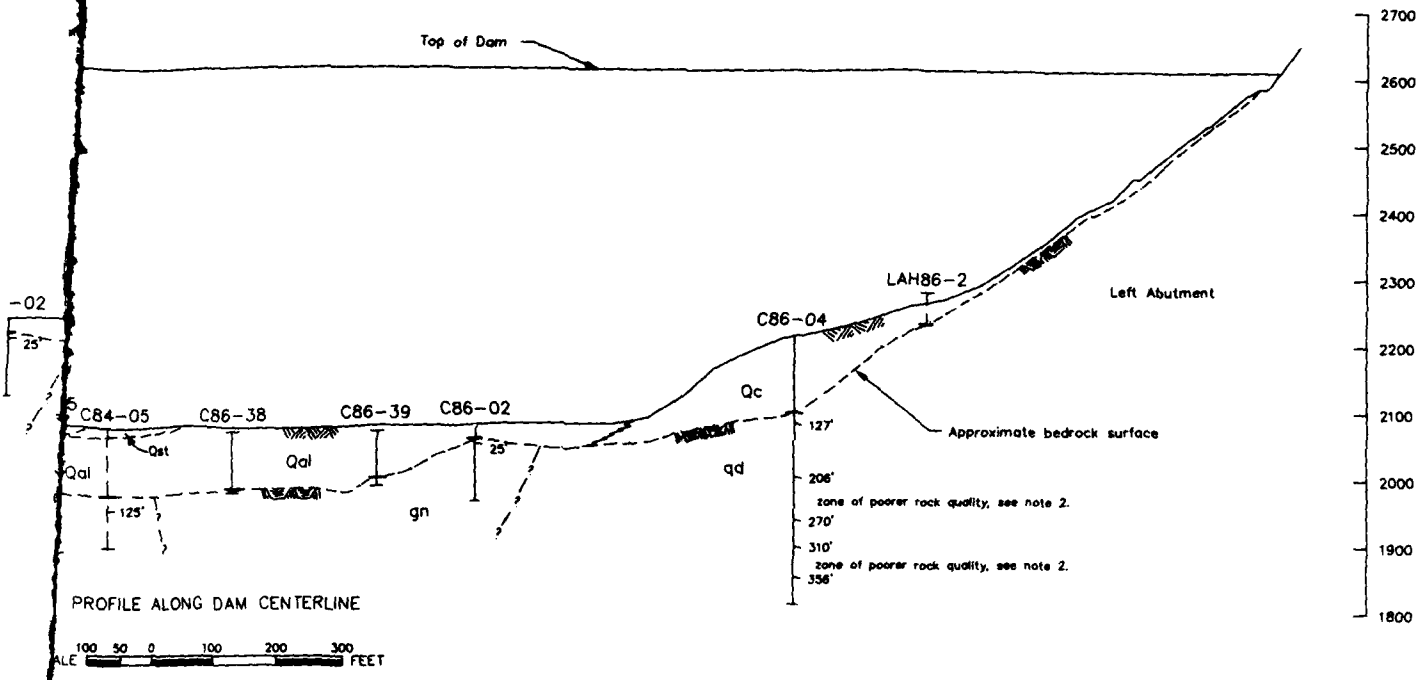
ENVIRONMENTAL
ENHANCEMENT
THRU ENGINEERING



LOCATION OF SECTIONS

SCALE 200 100 0 200 400 600 FEET

LUE ENGINEERING PAYS





LEGEND

C86-14

Exploratory core hole; dashed where projected

Bedrock/overburden contact

100' Depth where Rock Quality Parameters marginally improve; see note 2.

Inferred lithologic contact; queried where location questionable

Qal	Recent alluvium
Qc	Colluvium
Qst	Stream terrace
qd	Diorite
gn	Gneiss

NOTES:

1. See plates A-11 thru A-29 for Drill Logs of core holes.
2. See plates A-30 thru A-35 for *Rock Quality* Parameter Comparisons.
3. See plates A-43 and A-44 for geologic sections A thru D.
4. See plate A-103 for Abutment Excavation sections E and F.

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

EMBANKMENT
GEOLOGIC PROFILE AND PLAN

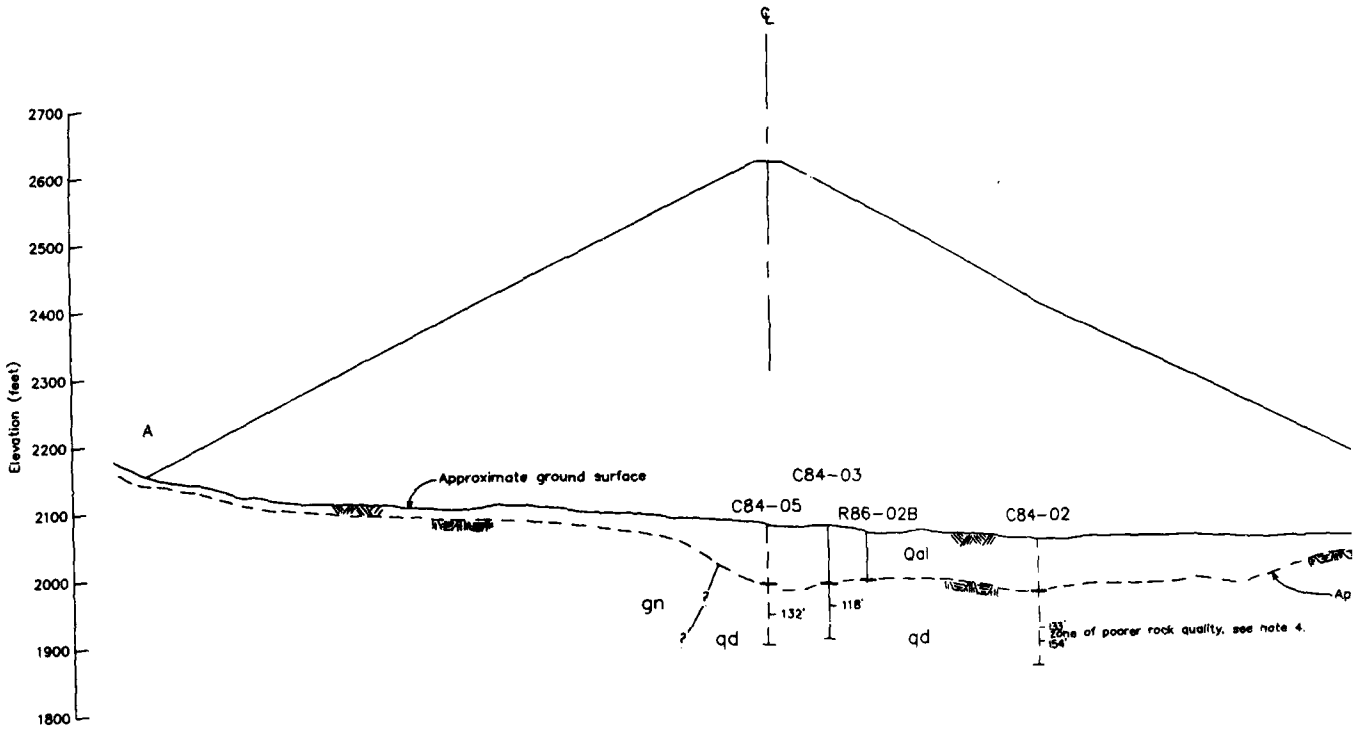
SUBMITTED BY: DATE APPROVED: SPEC. NO. SACW 09- -

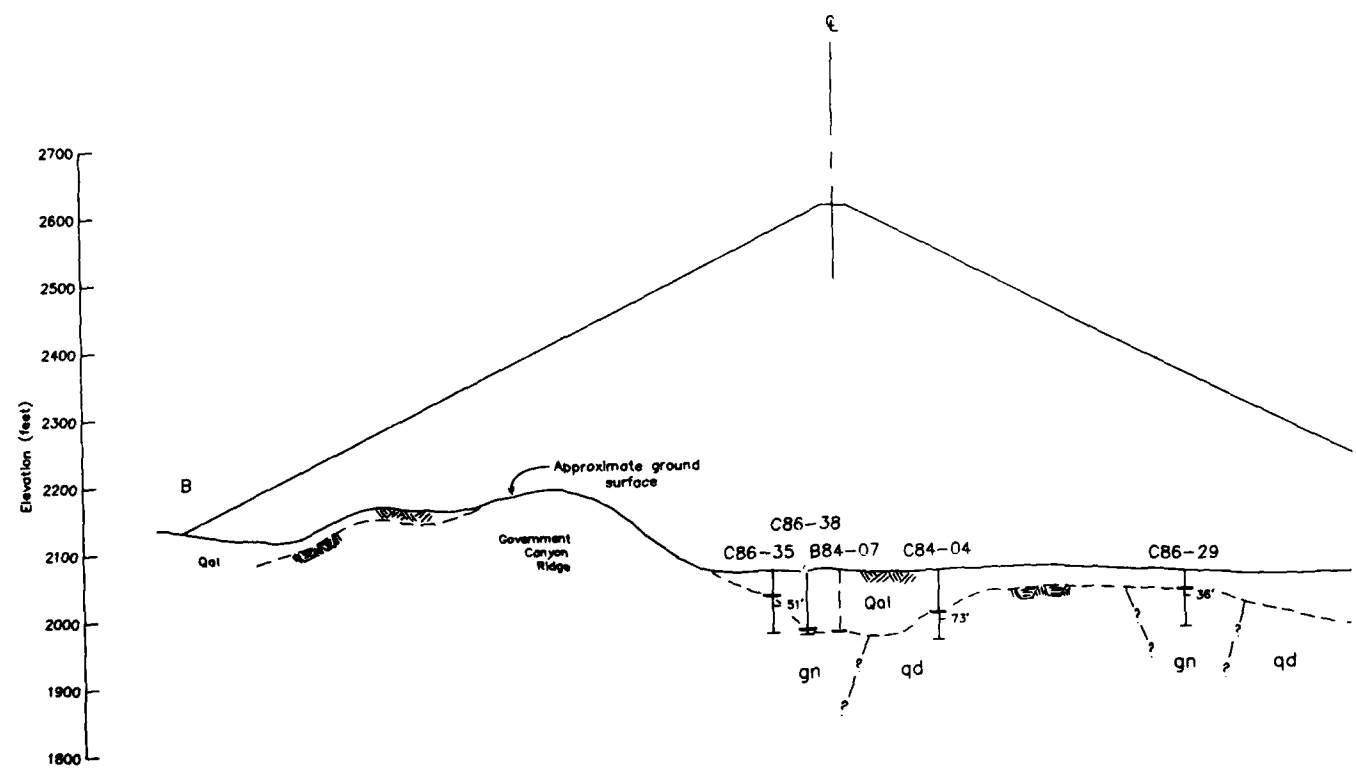
DISTRICT FILE NO.

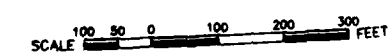
SHEET

PLATE A-42

SAFETY PAYS

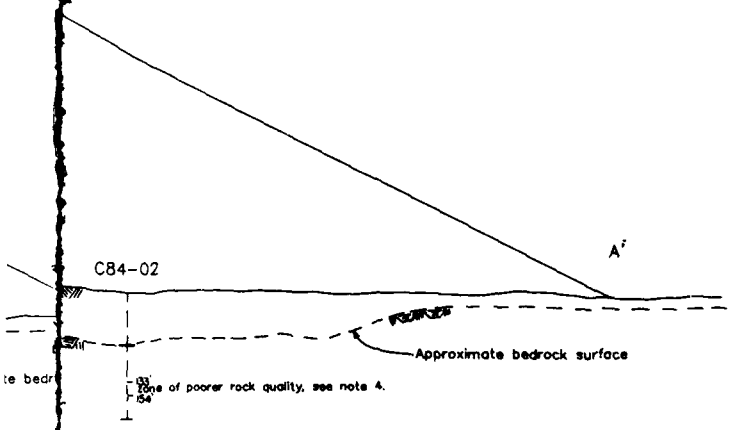


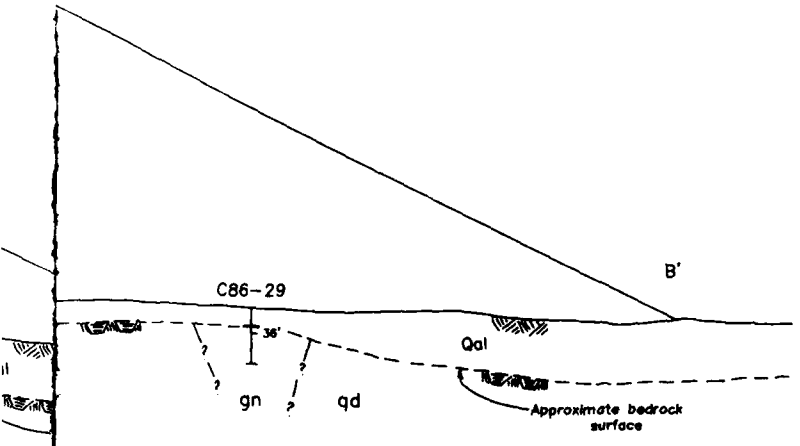




ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

ALUE ENGINEERING PAYS





300 FEET

NOTES:

1. See plate A-42 for location of section lines and legend.
2. See plate A-9 for locations of exploratory holes.
3. See plates A-11 thru A-17 for drill logs of core and rotary holes.
4. See plates A-30 thru A-35 for Rock Quality Parameter Comparisons.

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

EMBANKMENT
GEOLOGIC SECTIONS

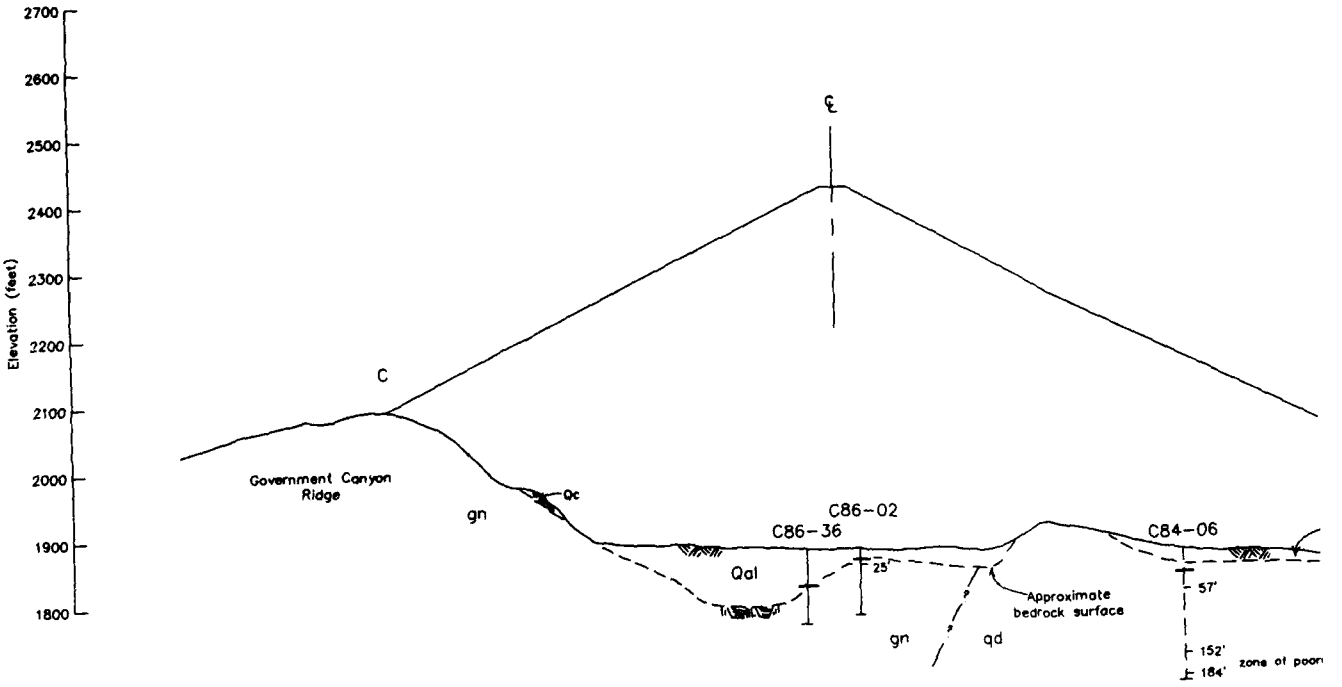
SUBMITTED BY: DATE APPROVED
CHIEF, ENGINEERING DIVISION

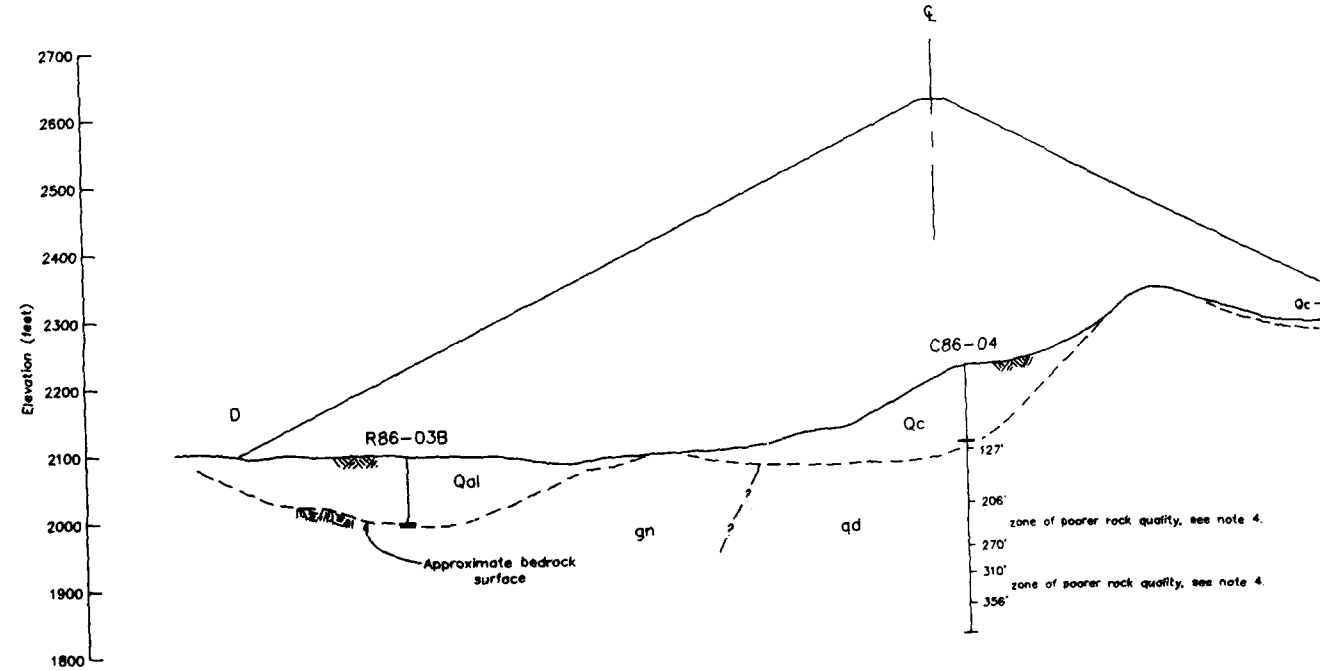
SPEC. NO. DACW09-____-B-____
DISTRICT FILE NO.

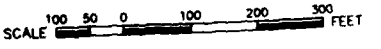
SHEET 1 OF 2 SHEETS

PLATE A-43

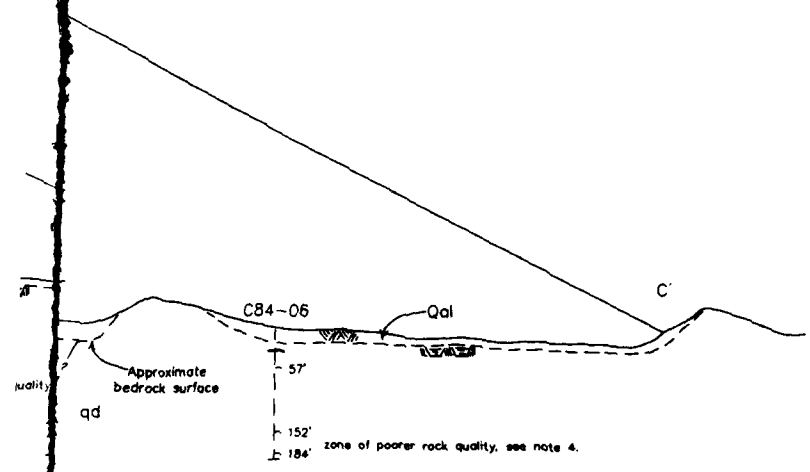
SAFETY PAYS

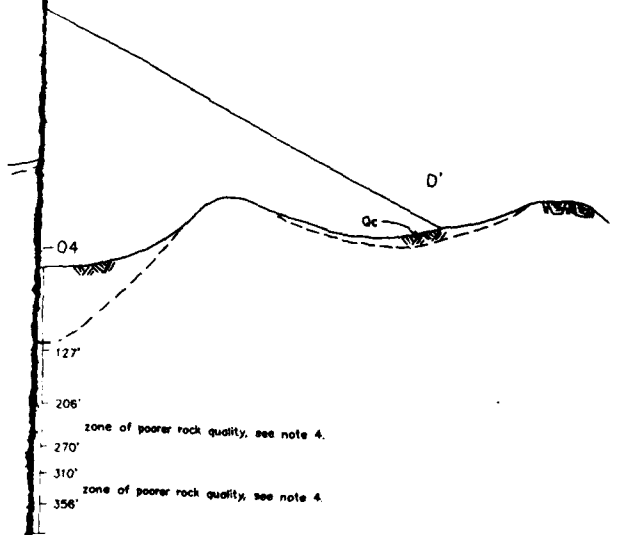

Elevation (feet)
2700
2600
2500
2400
2300
2200
2100
2000
1900
1800
℄
C
Government Canyon Ridge
gn
Qc
C86-02
C86-36
C84-06
Qal
25'
57'
Approximate bedrock surface
gn
qd
152'
184'
zone of poor


Elevation (feet)
2700
2600
2500
2400
2300
2200
2100
2000
1900
1800
℄
D
R86-03B
Qal
C86-04
Qc
127'
206'
zone of poorer rock quality, see note 4.
270'
310'
zone of poorer rock quality, see note 4.
356'
Approximate bedrock surface
gn
qd


SCALE
100 50 0 100 200 300
FEET

Y
ALUE ENGINEERING PAYS





0 300 FEET

NOTES:

1. See plate A-42 for location of section lines and legend.
2. See plate A-9 for locations of exploratory holes.
3. See plates A-11 thru A-17 for drill logs of core and rotary holes.
4. See plates A-30 thru A-35 for Rock Quality Parameter Comparisons.

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

EMBANKMENT
GEOLOGIC SECTIONS

SUBMITTED BY: DATE APPROVED:

SPEC. NO. DACW09-____-____

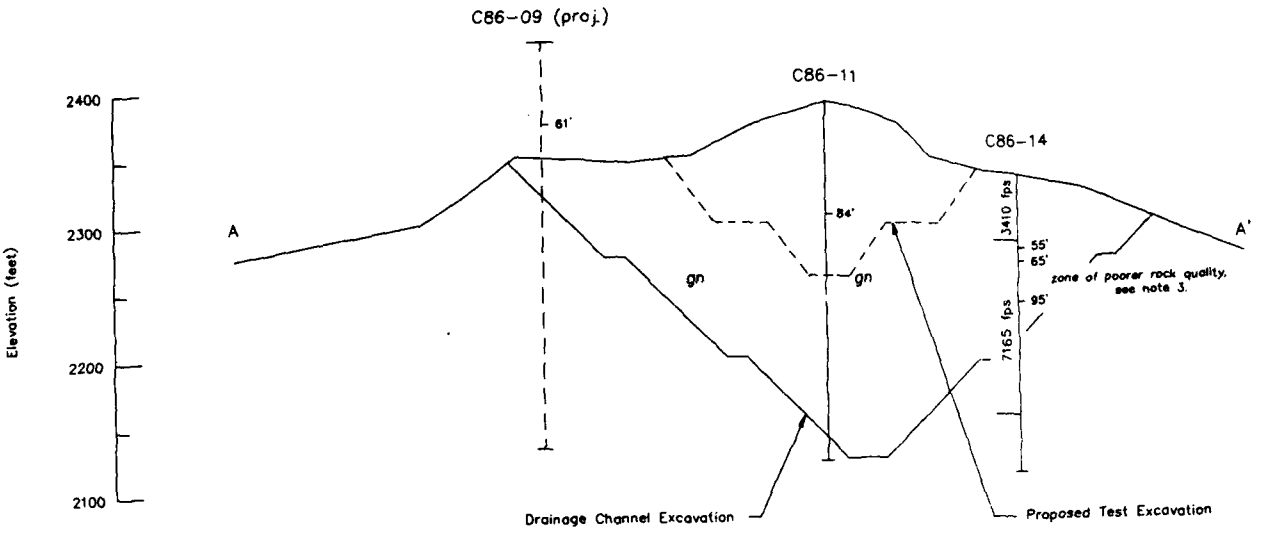
DISTRICT FILE NO.

SHEET 2 OF 2 SHEETS

TRACE SURVEY

PLATE A-44

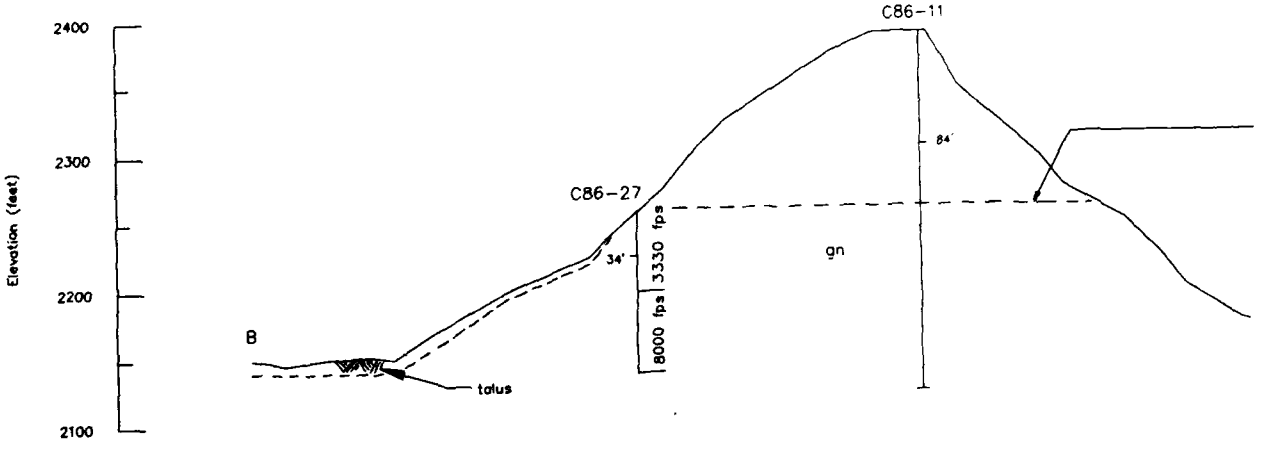
SAFETY PAYS



GOVERNMENT CANYON RIDGE

SECTION A-A'

SCALE 50 25 0 50 100 150 FEET



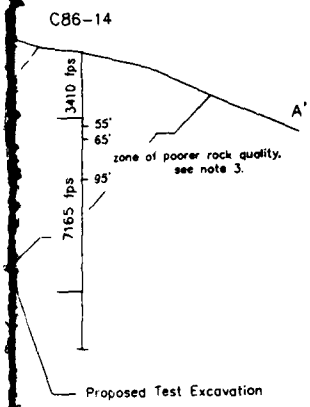
GOVERNMENT CANYON RIDGE

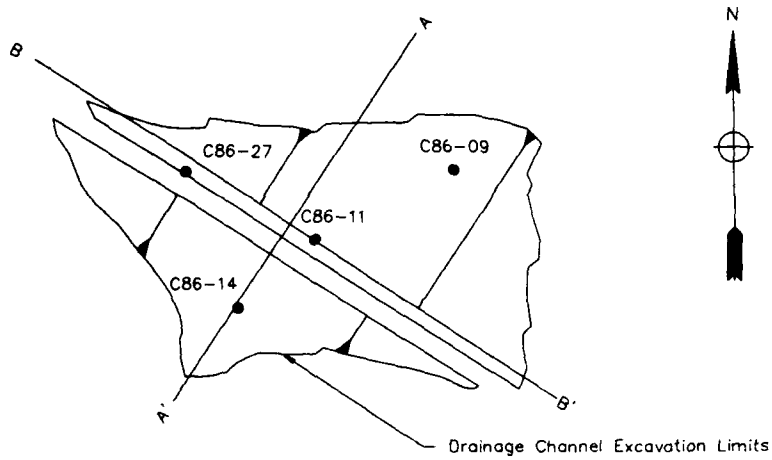
PROFILE B-B'

SCALE 50 25 0 50 100 150 FEET

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

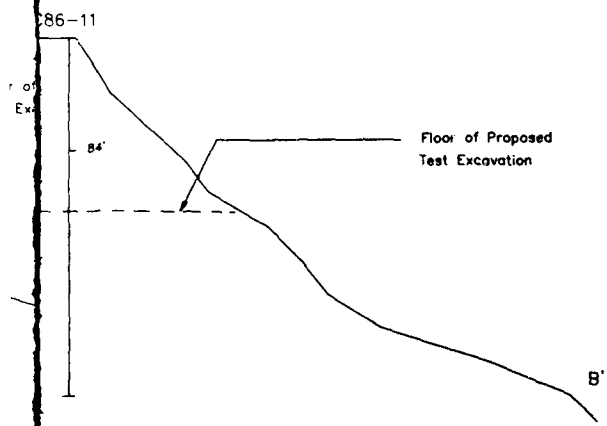
VALUE ENGINEERING PAYS



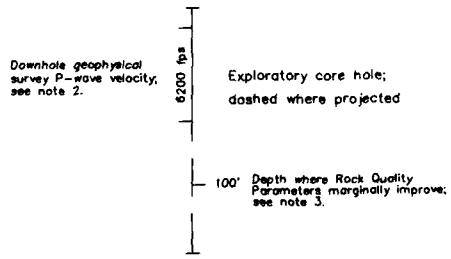


PLAN

SCALE 100 50 0 100 200 300 FEET



LEGEND



NOTES:

1. See plates A-24 and A-25 for *Drill Logs of core holes.*
2. See plate A-40 for Downhole Seismic Graphs.
3. See plate A-32 and plate A-34 for Rock Quality *Parameter* Comparisons.

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

GOVERNMENT CANYON RIDGE
GEOLOGIC PROFILE AND SECTION

SUBMITTED BY: DATE APPROVED:

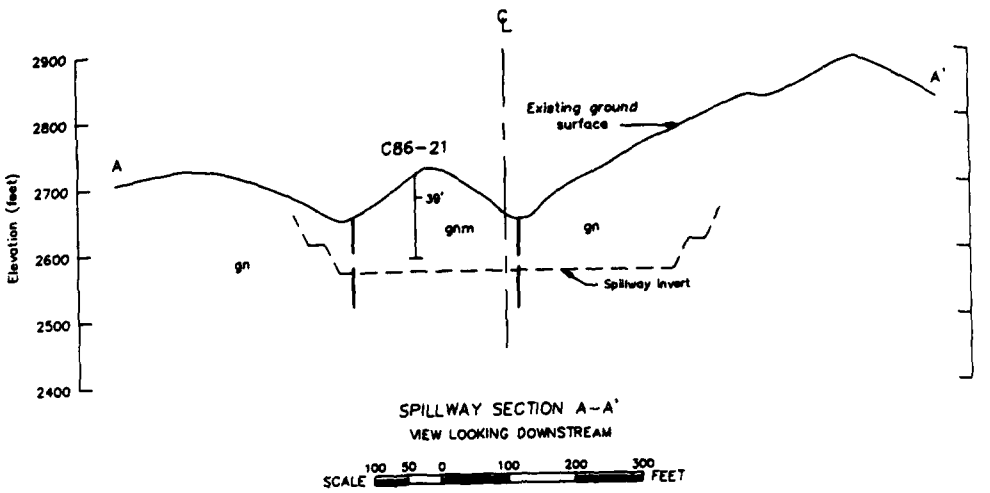
SPEC. NO. DACW09-____-B-____

DISTRICT FILE NO.

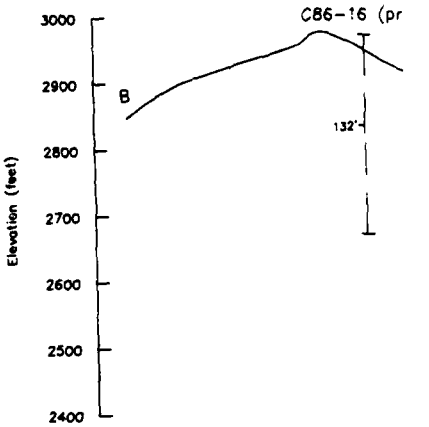
SHEET

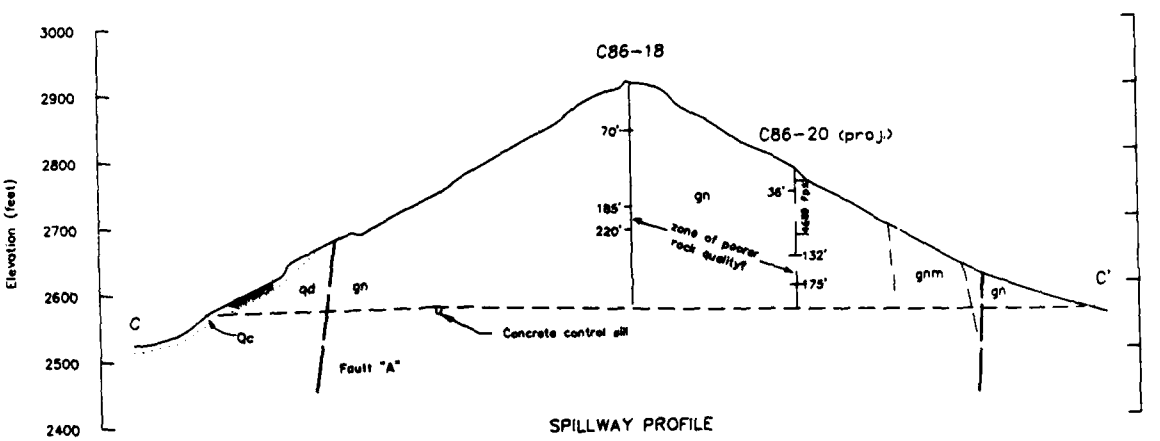
PLATE A-45

SAFETY PAYS

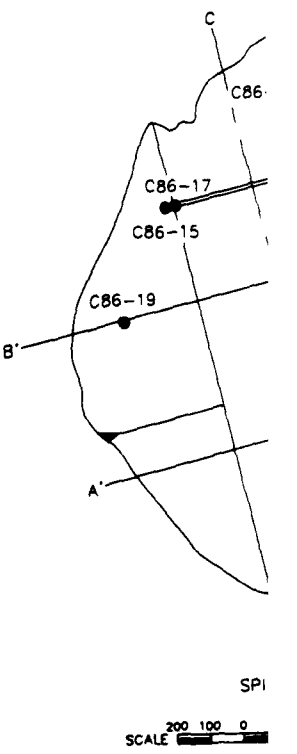


SPILLWAY SECTION A–A'
VIEW LOOKING DOWNSTREAM
SCALE 100 50 0 100 200 300 FEET



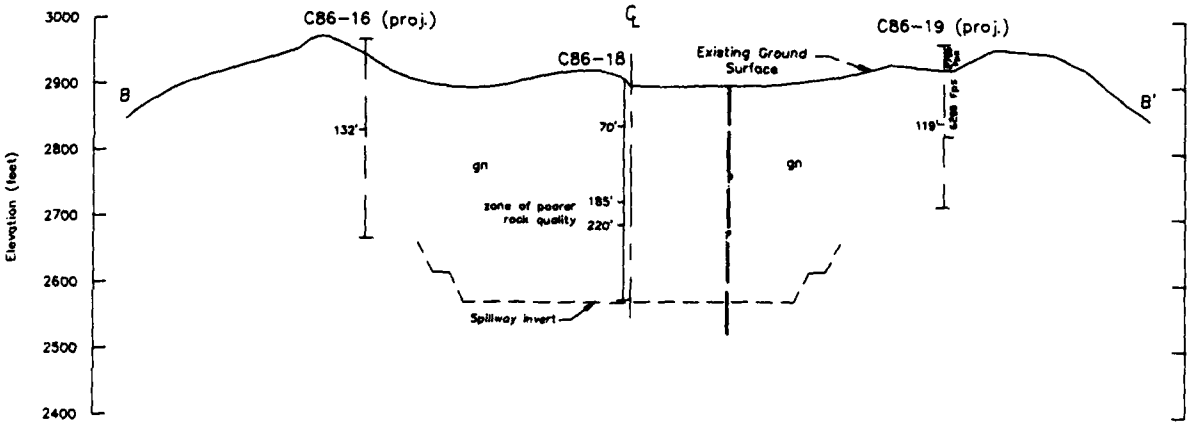


SPILLWAY PROFILE
ALONG CENTERLINE
SCALE 100 50 0 100 200 300 FEET



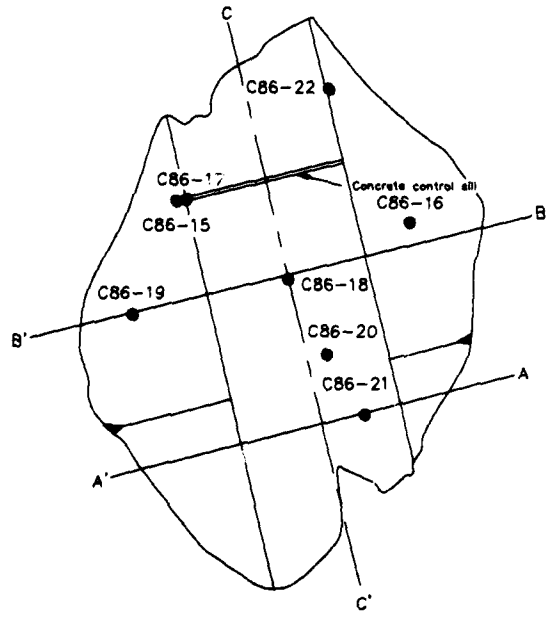
SPI
SCALE 200 100 0

VALUE ENGINEERING PAYS



SPILLWAY SECTION B-B'
VIEW LOOKING DOWNSTREAM

SCALE 100 50 0 100 200 300 FEET



SPILLWAY PLAN

SCALE 200 100 0 200 400 600 FEET

LEGEND

C86-16

Exploratory core hole; dashed where projected

Downhole geophysical survey P-wave velocity; see note 2.

Depth where Rock Quality Parameters marginally improve see note 3. 100'

fault

lithologic contact

Qc/rock contact

gn	gneiss
gnm	mafic-rich gneiss
qd	diorite
Qc	colluvium

NOTES:

1. See plates A-26 thru A-29 for Drill Logs of core holes.
2. See plate A-40 for Downhole Seismic Graphs.
3. See plates A-32 thru A-34 for Rock Quality Parameter Comparisons.

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

SPILLWAY
GEOLOGIC SECTIONS AND PROFILE

SUBMITTED BY: DATE APPROVED:

SPEC. NO. DACW 09- ____ B- ____

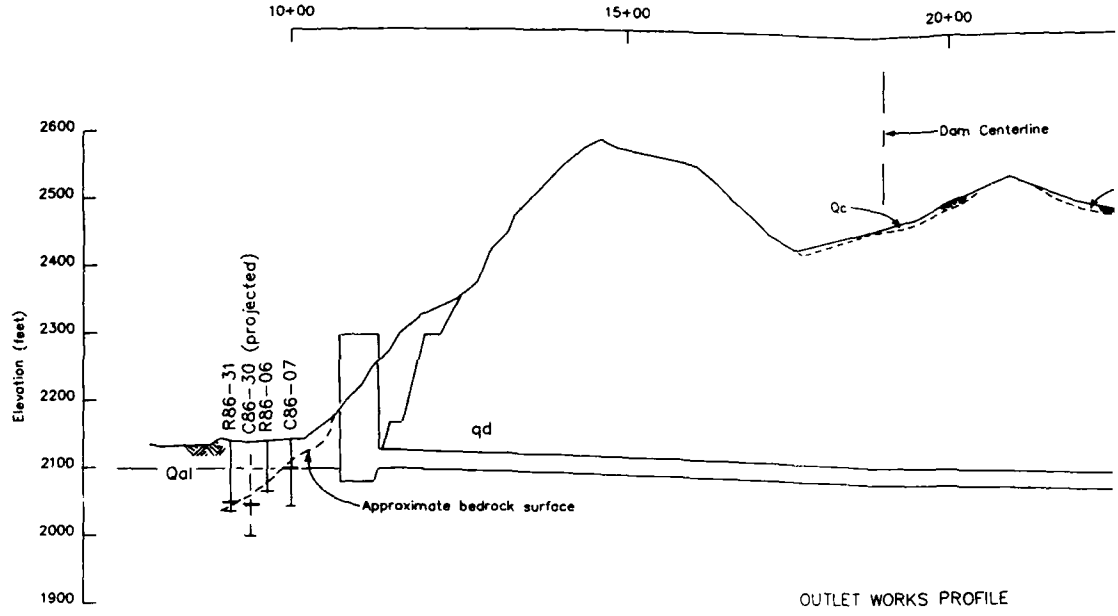
DISTRICT FILE NO.

SHEET

TOTAL SHEETS

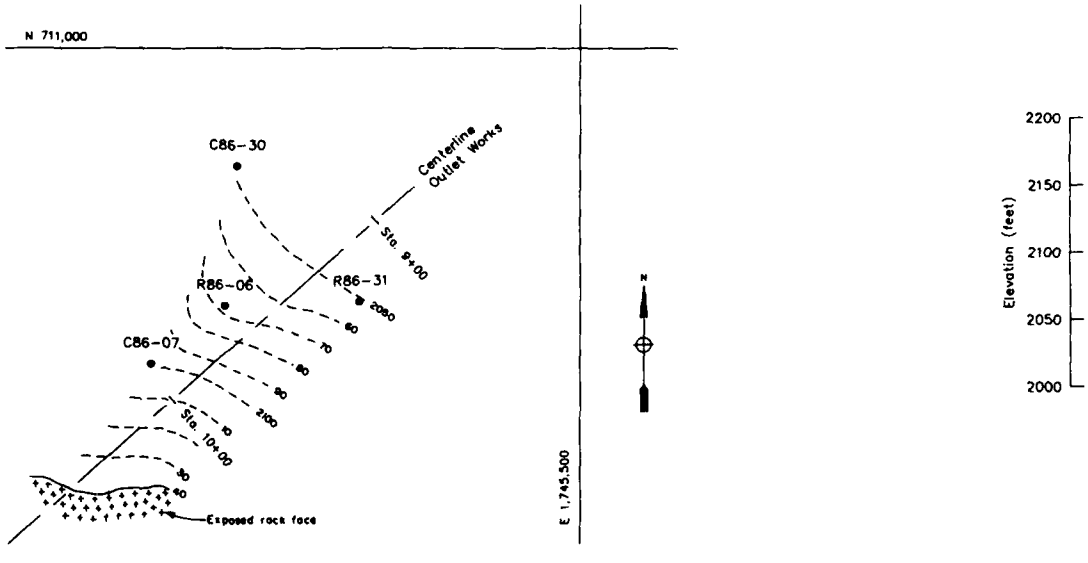
PLATE A-46

SAFETY PAYS



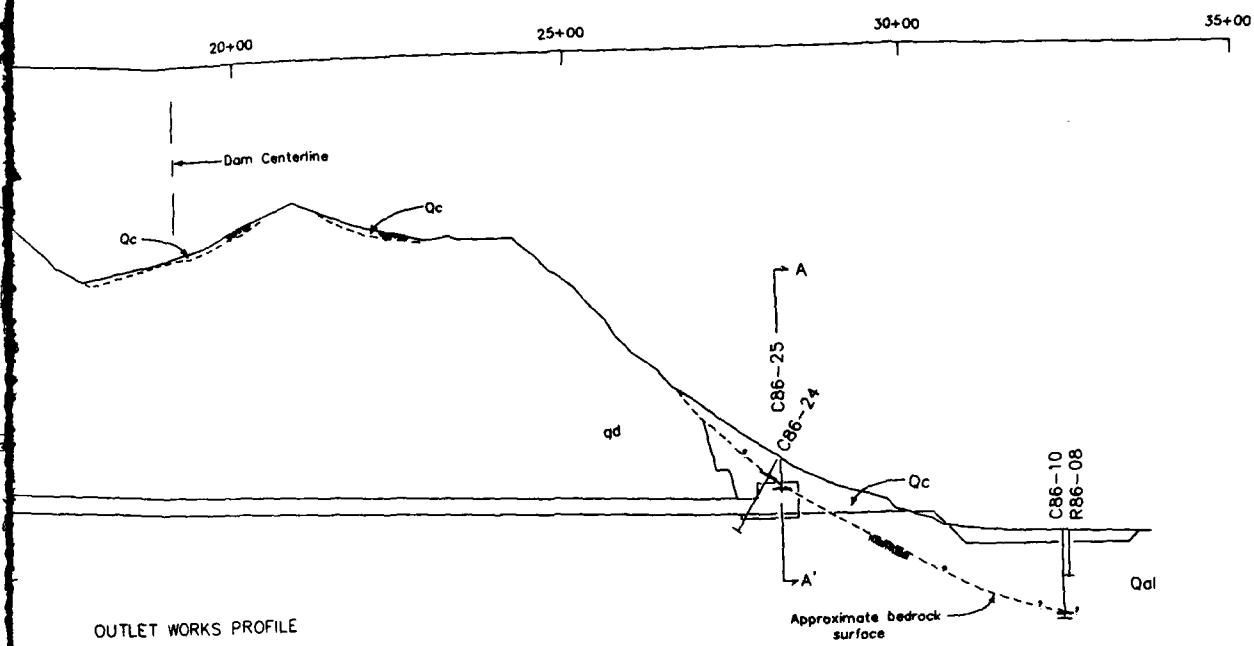
OUTLET WORKS PROFILE

SCALE 100 50 0 100 200 300 FEET



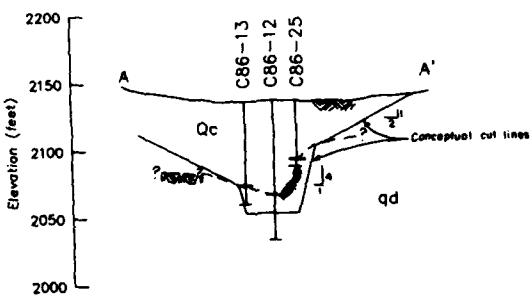
INLET PORTAL
BEDROCK CONTOURS

SCALE 25 0 25 50 75 FEET



OUTLET WORKS PROFILE

SCALE 100 50 0 100 200 300 FEET



DOWNSTREAM PORTAL
SECTION A-A'

SCALE 50 25 0 50 100 150 FEET

NOTES:

1. Structural features shown are conceptual for downstream control as presented by Portland District in Appendix B. They are intented to show the general relationship of the geologic conditions to the outlet works. This in no way implies acceptance of downstream control as the most favorable.
2. See plate A-9 for Locations of Exploratory Holes.
3. See plates A-21 and A-22 for Drill Logs of Outlet Works exploratory holes.

SYMBOL	DESCRIPTION	DATE	APP

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

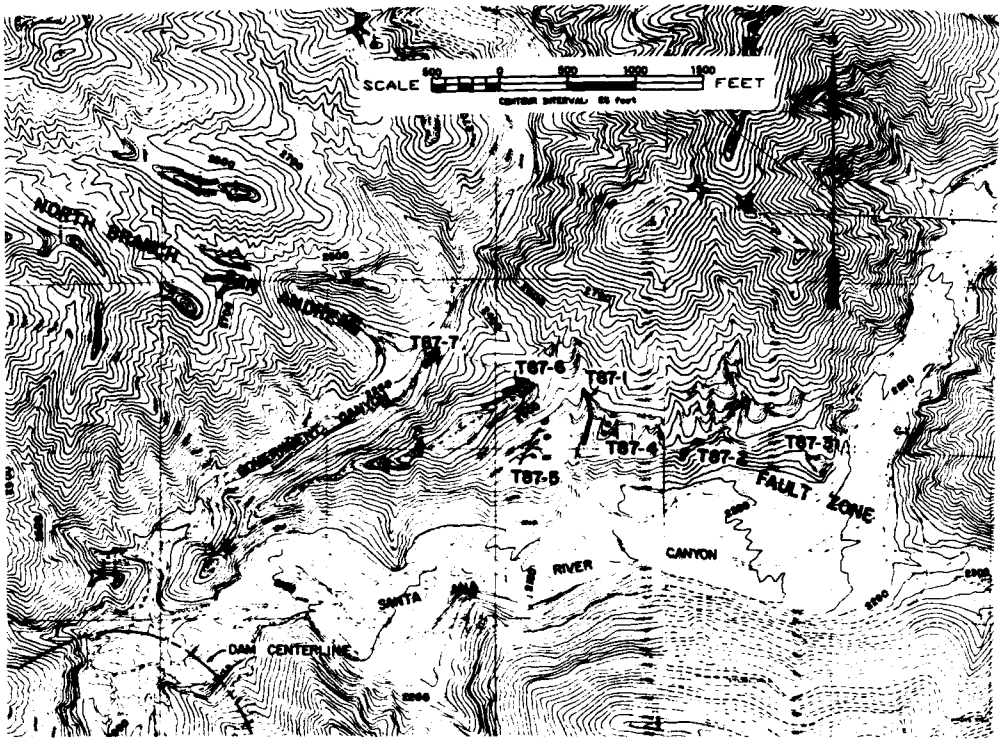
SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

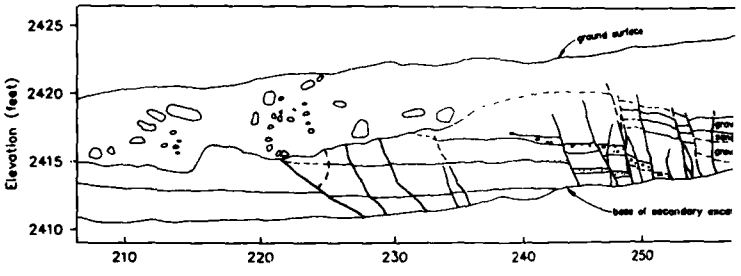
OUTLET WORKS
GEOLOGIC PROFILE
AND SUBSURFACE BEDROCK CONTOUR

DESIGNED BY:
DRAWN BY:
CHECKED BY:
SUBMITTED BY:
DATE APPROVED:
CHIEF, BRANCH
SPEC. NO DACW09-____-____
DISTRICT FILE NO.

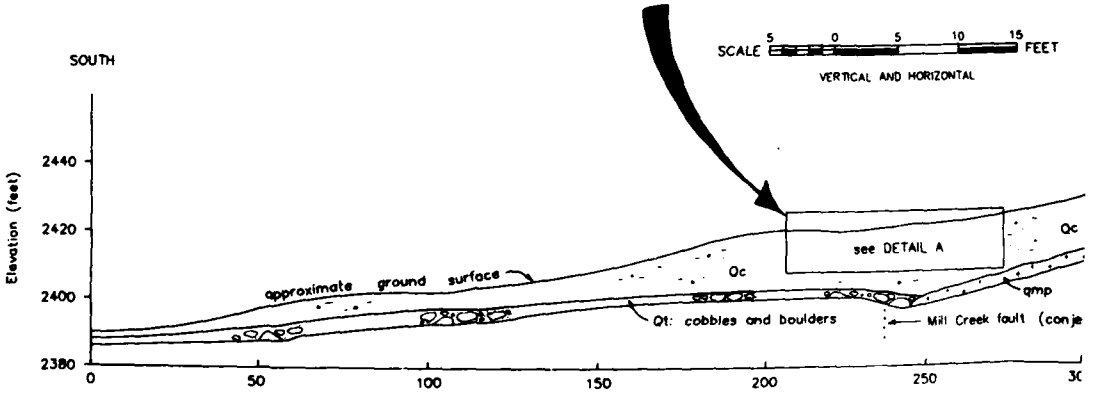
PLA



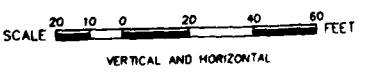
DETAIL A



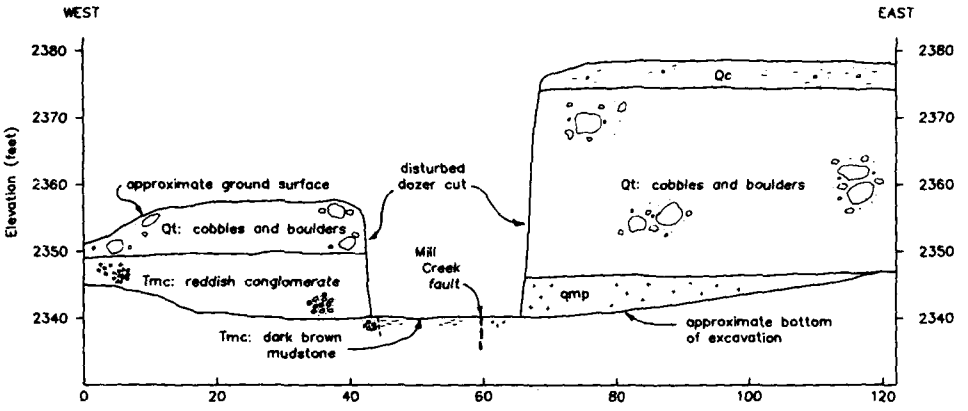
Detail of T87-1 based on interpretation by Ray Weldon



T87-1



ALUE ENGINEERING PAYS

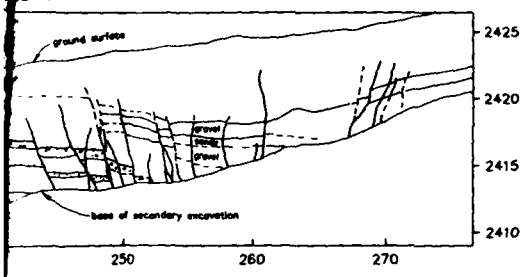


T87-2

SCALE 10 5 0 10 20 30 FEET

VERTICAL AND HORIZONTAL

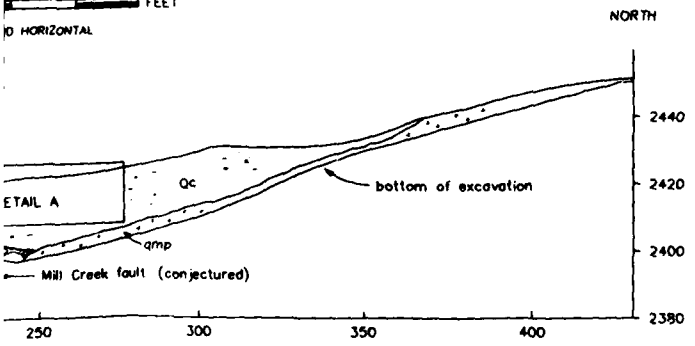
L A



nterpretation by Ray Weldon

5 10 15 FEET

0 HORIZONTAL



60 FEET

NOTES:

1. See plate A-6 for legend and description of geologic units.
2. See section 5 of this appendix for discussion of fault investigations.
3. T87-3, T87-4 and T87-5 were soil pits used to collect samples for soil chronology studies and logs are not shown.
4. T87-1 and T87-2 were excavated with a D-8 dozer.
5. Dibblee (1964) equates the North Branch San Andreas fault with Allen's (1957) Mill Creek fault.

SYMBOL	DESCRIPTION	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

NORTH BRANCH FAULT INVESTIGATIONS
LOCATION AND LOGS

SUBMITTED BY: DATE APPROVED:
CHIEF, DIVISION

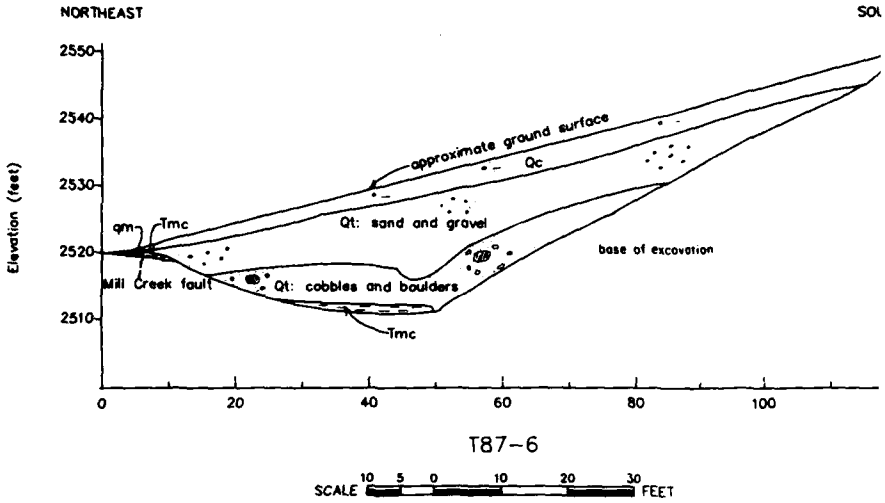
SPEC. NO. DACW09-___-___
DISTRICT FILE NO.

SHEET

PLATE A-

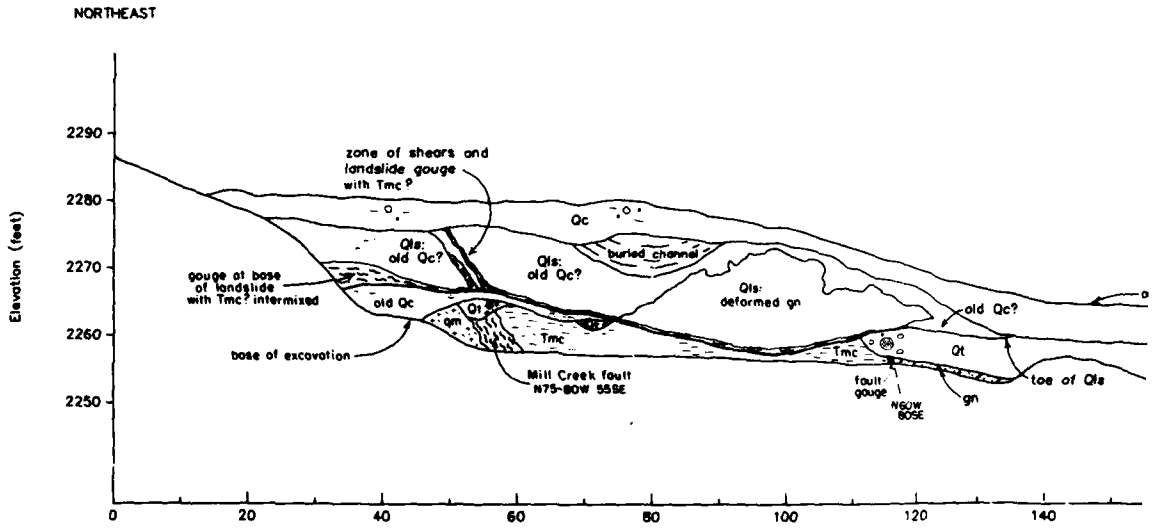
SAFETY PAYS

ENVIRONMENTAL
ENHANCEMENT
THRU ENGINEERING



T87-6

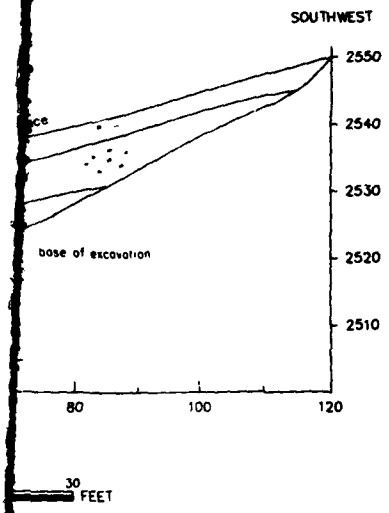
SCALE 10 5 0 10 20 30 FEET

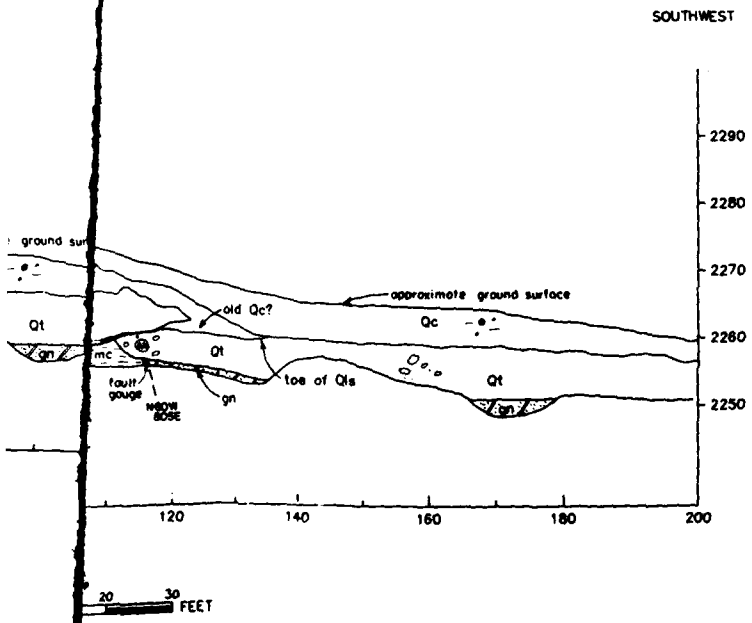


T87-7

SCALE 10 5 0 10 20 30 FEET

VALUE ENGINEERING PAYS





NOTES:

1. See plate A-48 for location of trenches.
2. See plate A-6 for legend and description of geologic units.
3. See section 5 of this appendix for description of fault investigations.
4. T87-6 and T87-7 were excavated with a D-8 dozer. Additional excavation in the bottom of T87-7 was done with a backhoe.
5. Dibblee (1964) equates the North Branch San Andreas fault with Allen's (1957) Mill Creek fault.

SYMBOL	DESCRIPTION	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS
SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SEVEN OAKS DAM

NORTH BRANCH FAULT INVESTIGATIONS
TRENCH LOGS

SUBMITTED BY: DATE APPROVED:
SPEC. NO. DACW09- - -
DISTRICT FILE NO.

PLATE A

SAFETY PAYS

TT84-1
EL. 2070 ±

VISUAL LOG

DEPTH	VISUAL LOG	DESCRIPTION
1.0'	SM	SILTY SAND; dark grey, damp, loose, fine grained sand
7.0'	SP	SAND; light brown, medium dense, fine to medium grained sand.
15.0'	SW	GRAVELLY SAND; coarse grained sand, abundant gravel to 3", abundant cobbles to 10", boulders to 7'
▼21.0' 21.5'	SP	SAND; dark yellowish brown, moist, dense, medium grained sand, scattered gravel to 3", cobbles to 10".

LARGE-SCALE DENSITY TEST RESULTS

DEPTH	LOG	MC	γ_d	-3"	#4	#200	DESCRIPTION
0.5							SILTY SAND.
2.5'	GP	3	118	93	50	1	SANDY GRAVEL.
4.0'							
6.0'	GP	2	123	88	49	2	SANDY GRAVEL.
10.0'							
12.0'	GP	2	133	89	34	1	SANDY GRAVEL.
14.0'							
16.0'	GP	3	114	72	31	0	SANDY GRAVEL.
19.5'							
▼21.0' 21.5'	GP	10	136	60	30	1	SANDY GRAVEL.

TT84-3
EL. 2055 ±

VISUAL LOG

DEPTH	VISUAL LOG	DESCRIPTION
6.0'	SW	GRAVELLY SAND; brown to light brown, damp, medium dense, coarse grained sand, gravel to 3", cobbles to 10", boulders to 2'.
9.0'	SP	SAND; yellowish brown, moist, medium dense, coarse grained sand, scattered gravel to 1".
15.0'	GW	SANDY GRAVEL; yellowish brown, moist, dense, coarse grained gravel mostly 3", coarse grained sand matrix, cobbles to 10", abundant boulders to 4½'.
▼17.0'	SP	SAND; yellowish brown, wet, dense, medium grained sand, scattered gravel and cobbles.

LARGE-SCALE DENSITY TEST RESULTS

DEPTH	LOG	MC	γ_d	-3"	#4	#200	DESCRIPTION
2.5'							GRAVELLY SAND.
4.5'	GP	3	137	64	27	0	SANDY GRAVEL.
8.0'							
10.0'	SP	3	112	81	60	1	GRAVELLY SAND.
15.0'							
▼17.0'	GP			92	48	2	SANDY GRAVEL; no large-scale density test.

NOTES:

1.
2.
3. THE DEPTH TO GROUNDWATER IS APPROXIMATE AND VARIES SEASONALLY AND YEARLY.
4. THE LOG OF TEST TRENCH INDICATES THE GEOTECHNIC CONDITION AT THAT LOCATION AT THAT PARTICULAR T THE STRATIFICATION LINES SHOWN ON THE LOGS REPR THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES AND TRANSITION MAY BE GRADUAL OR ABRUPT.
5. TEST TRENCHES WERE TERMINATED AT A DEPTH WHERE WATER WAS ENCOUNTERED.

SCALE

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

VALUE ENGINEERING PAYS

TT 84-2
EL. 2075 ±

VISUAL LOG

DEPTH	VISUAL LOG	DESCRIPTION
2.0'	SW	GRAVELLY SAND; light brown, damp, medium dense, coarse grained sand, gravel to 3", scattered cobbles to 8".
3.5'	SP	SAND; brown, moist, some gravel to ½"
9.5'	GW	SANDY GRAVEL; brown, moist, dense, gravel to 3", abundant cobbles to 8", some boulders to 15".
15.6'	SP	SAND; brown, moist, dense, medium grained sand, minor layer of 3" gravel, abundant boulders to 4'.
▽20.6'	SW	SILTY SAND; dark yellowish brown, moist to saturated, dense, coarse grained sand, trace of clay, gravel to 3", scattered cobbles to 10".
22.0'		brown, saturated, medium grained, gravel to 3", cobbles to 8".

LARGE-SCALE DENSITY TEST RESULTS

DEPTH	LOG	MC	γ_d	-3"	#4	#200	DESCRIPTION
2.0'							GRAVELLY SAND.
4.0'	SP	2	130	99	71	2	GRAVELLY SAND.
7.0'							
9.0'	SP	2	109	100	99	4	SAND.
13.0'							
15.0'	GP	2	136	60	40	1	SANDY GRAVEL.
20.0'							
20.6'	SP			93	58	3	GRAVELLY SAND; no large-scale density test.
▽22.0'							

NOTES:

1.
2.
3. THE DEPTH TO GROUNDWATER IS APPROXIMATE AND VARIES SEASONALLY AND YEARLY.
4. THE LOG OF TEST TRENCH INDICATES THE GEOTECHNICAL CONDITION AT THAT LOCATION AT THAT PARTICULAR TIME. THE STRATIFICATION LINES SHOWN ON THE LOGS REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL OR ABRUPT.
5. TEST TRENCHES WERE TERMINATED AT A DEPTH WHERE GROUNDWATER WAS ENCOUNTERED.

LEGEND

TT 84-3	YEAR AND NUMBER OF TEST TRENCH.
MC	FIELD MOISTURE CONTENT IN PERCENT OF DRY WEIGHT.
γ_d	FIELD DRY UNIT WEIGHT, POUNDS PER CUBIC FOOT.
-3"	PERCENT OF MATERIAL BY WEIGHT PASSING 3 INCH SIEVE.
-#4	PERCENT OF MATERIAL BY WEIGHT PASSING NO.4 SIEVE.
-#200	PERCENT OF MATERIAL BY WEIGHT PASSING NO.200 SIEVE.
▽	GROUNDWATER.

SCALE 5 0 5 10 FEET

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM
FOUNDATION
SOIL LOGS

SUBMITTED BY: DATE APPROVED:

SPEC. NO. DACW09-____-____

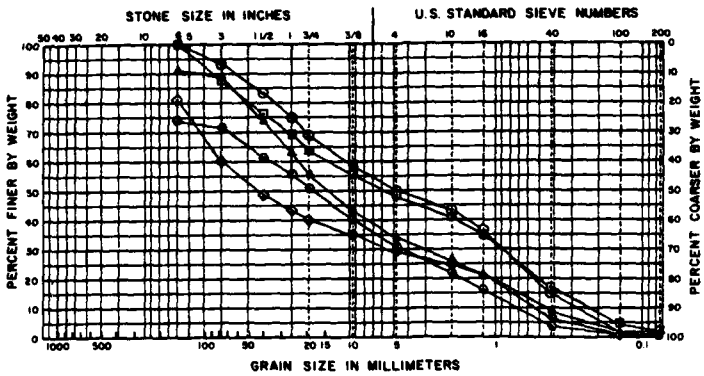
DISTRICT FILE NO.

SHEET

SAFETY PAYS

PLATE A-50

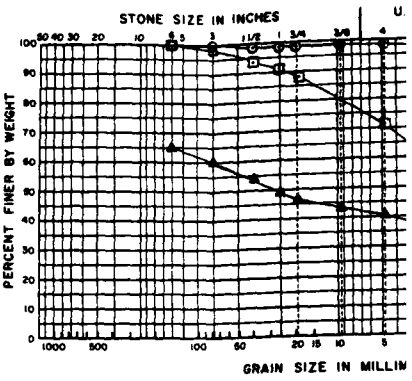
TP-1



SYMBOL	DEPTH (FEET)
○	0.5
□	4.2
△	10.0
○	14.0
◇	19.5

GRADATION CURVES

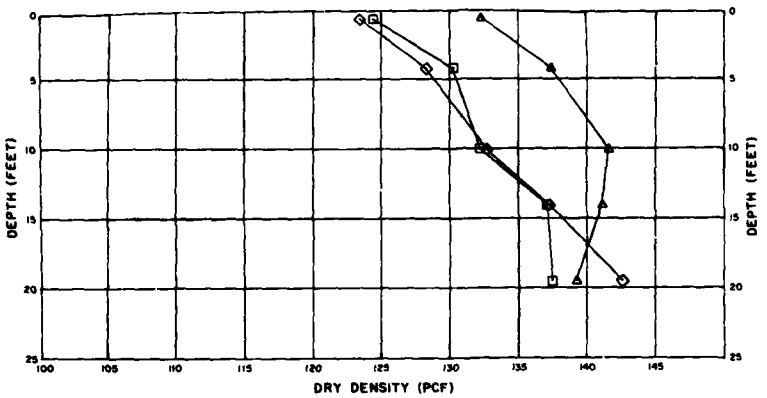
TP-2



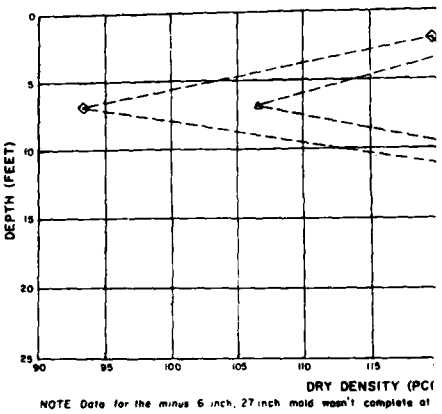
SYMBOL DEPT

□
○
△

GRADATION CURV

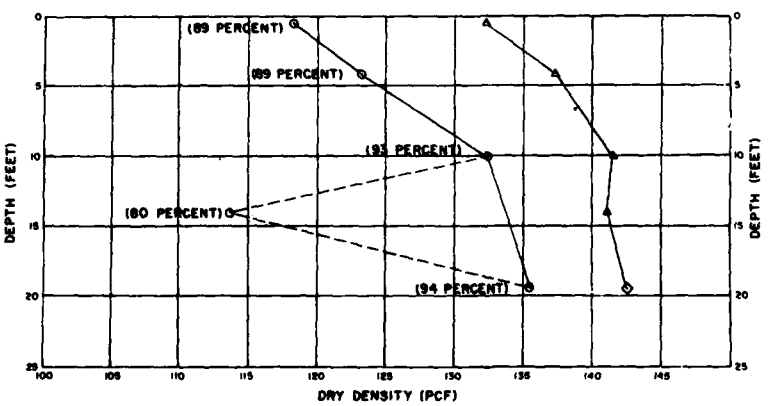


ADJUSTED MAXIMUM DRY DENSITY

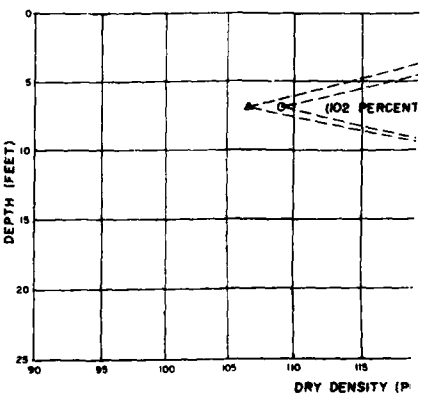


NOTE Data for the minus 6 inch, 27 inch mold wasn't complete at

ADJUSTED MAXIMUM DR



ADJUSTED MAXIMUM AND IN-SITU DRY DENSITY COMPARISON

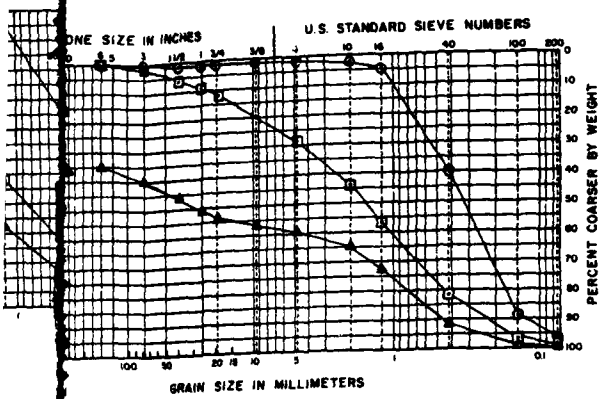


ADJUSTED MAXIMUM AND IN-SITU D

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

ALUE ENGINEERING PAYS

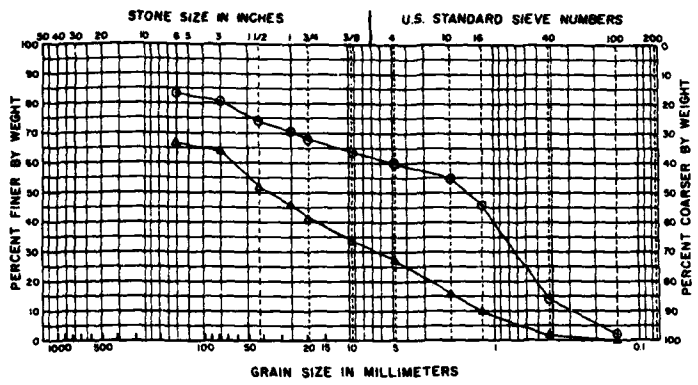
TP-2



SYMBOL	DEPTH (FEET)
□	2.0
○	6.8
△	13.0

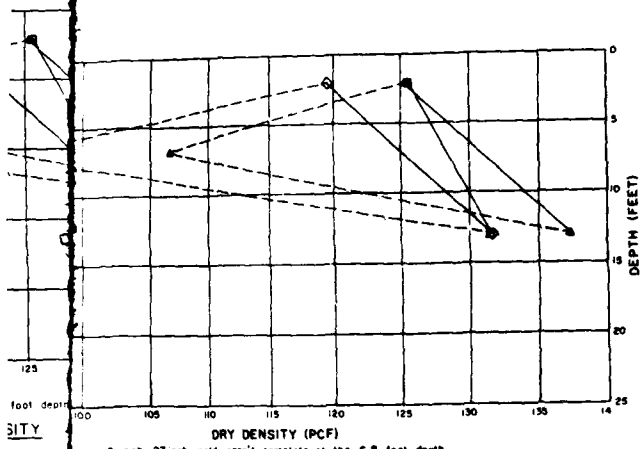
GRADATION CURVES

TP-3



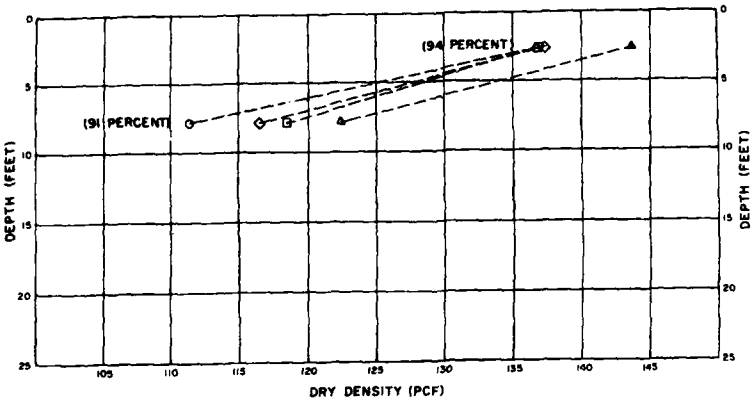
SYMBOL	DEPTH (FEET)
△	2.6
○	7.8

GRADATION CURVES



inus 6 inch, 27-inch mold wasn't complete at the 6.8 foot depth.

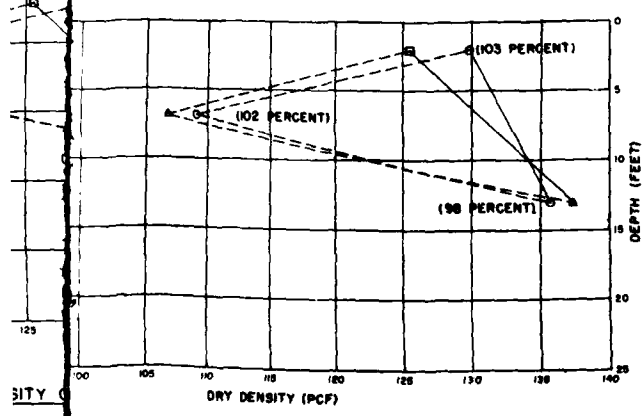
ADJUSTED MAXIMUM DRY DENSITY



ADJUSTED MAXIMUM AND IN SITU DRY DENSITY

NOTES:

1. MAXIMUM VIBRATED DENSITY TESTS ADJUSTED TO FIELD GRADATIONS
   - ⊡ MINUS 6-INCH MATERIALS, 27-INCH MOLD, DWR METHOD.
   - ◈ MINUS 3-INCH MATERIALS, 27-INCH MOLD, DWR METHOD.
   - ▲ MINUS 1 1/2-INCH MATERIALS, 11-INCH MOLD, ASTM D 4253-83.
2. IN-SITU DRY DENSITY AND RELATIVE COMPACTION
   - ⊙ LARGE-SCALE IN-SITU DRY DENSITY.
   - (93 PERCENT) RELATIVE COMPACTION OR PERCENT COMPACTION, RATIO OF IN-SITU DRY DENSITY AND ADJUSTED MAXIMUM DRY DENSITY INDICATED.



ED MAXIMUM AND IN-SITU DRY DENSITY COMPARISON

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

FOUNDATION MATERIALS
GRADATION CURVES AND
ADJUSTED MAXIMUM DRY DENSITIES

SUBMITTED BY: DATE APPROVED:
CHIEF, BRANCH

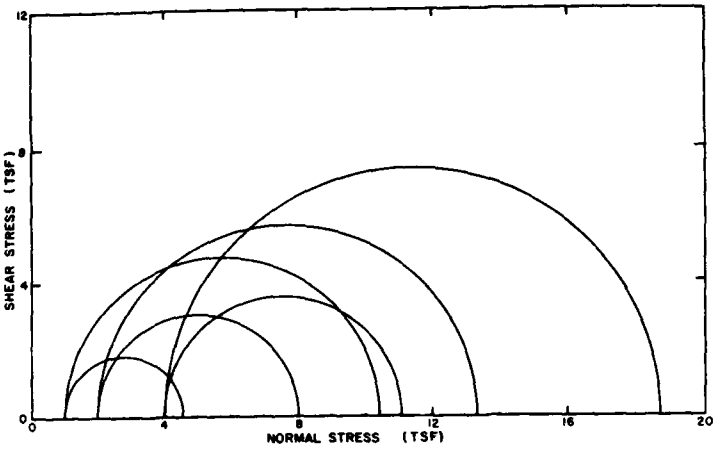
SPEC. NO. DACW09-____-____
DISTRICT FILE NO.

SHEET

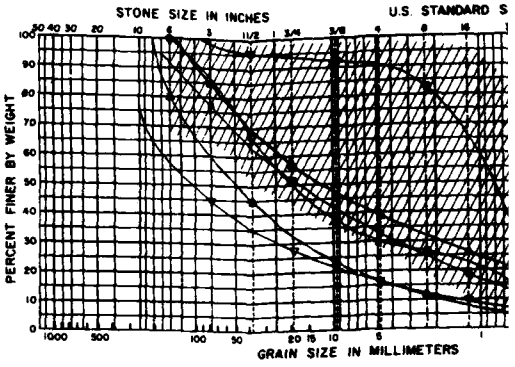
PLATE A-51

SAFETY PAYS

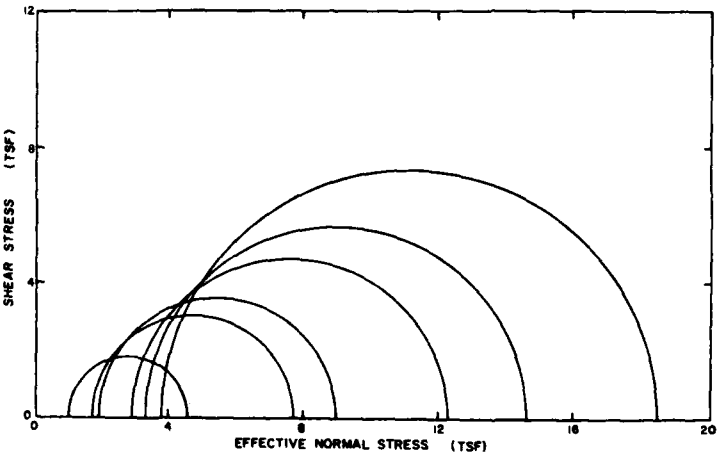
UPPER SANTA ANA RIVER DAMSITE



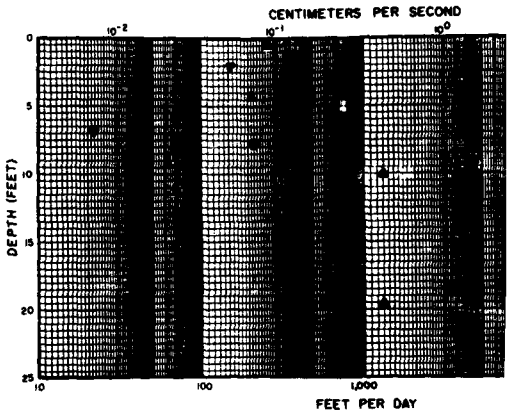
CONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TESTS



DENSITY AND GRADATION COMPARISO

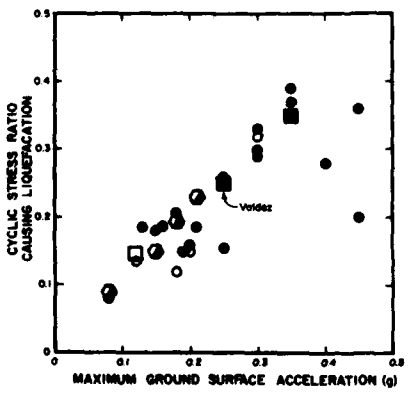


CONSOLIDATED-DRAINED TRIAXIAL COMPRESSION TESTS

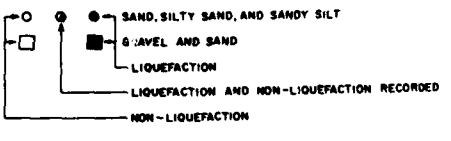


CALCULATED PERMEABILITY[1]

▲ - TP-1
▼ - TP-2
● - TP-3



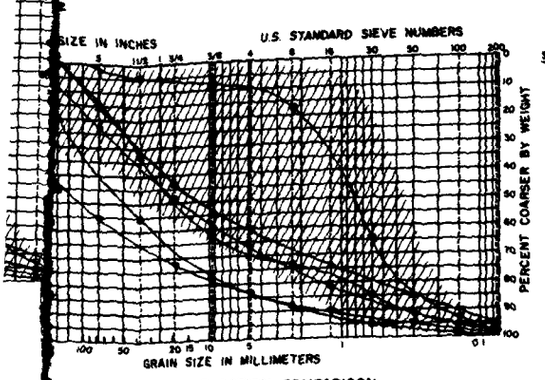
PLOT OF CASE HISTORY DATA[2]



ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

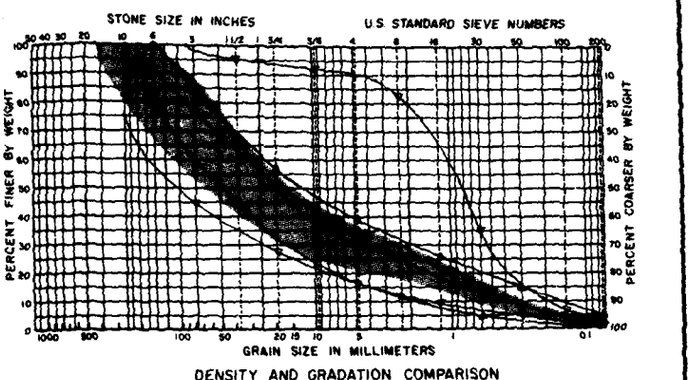
VALUE ENGINEERING PAYS

LYTLE CREEK DAMSITE

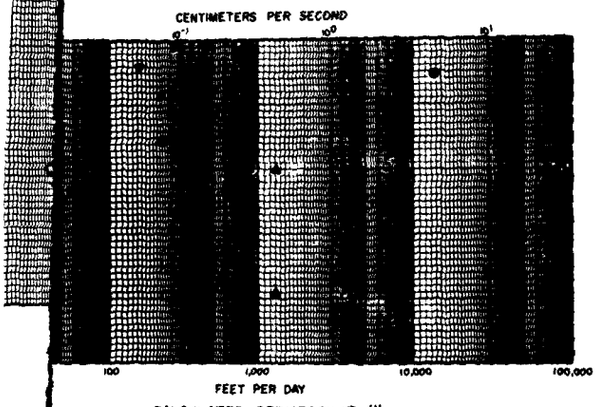


DENSITY AND GRADATION COMPARISON

SYMBOL	SITE	AVERAGE DRY DENSITY (PCF)
△	CUCAMONGA EMBANKMENT	138
□	SAN ANTONIO DAM EMBANKMENT	139
○	SANTA FE DAM EMBANKMENT	136
▽	MENTONE DAMSITE FOUNDATION	135
///	UPPER SANTA ANA RIVER DAMSITE FOUNDATION	130
▓	LYTLE CREEK DAMSITE FOUNDATION	136

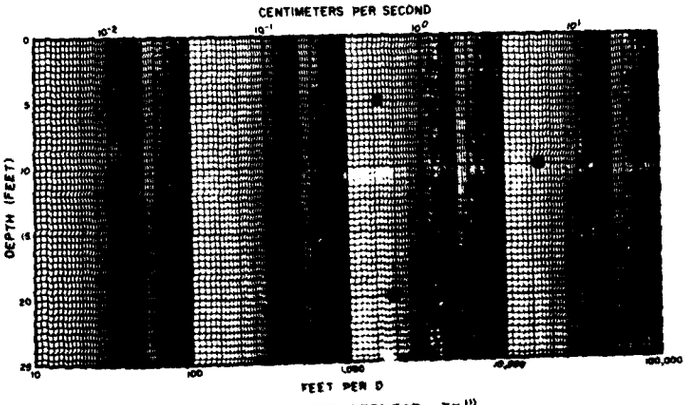


DENSITY AND GRADATION COMPARISON



CALCULATED PERMEABILITY[1]

▲ - TP-1
▼ - TP-2
● - TP-3



CALCULATED PERMEABILITY[1]

■ TT85-1
◆ TT85-2

NOTES:

1. PERMEABILITY, K, CALCULATED USING THE FOLLOWING EQUATION:
   $K(CM/SEC) = 77 (D_{20})^{2.32}$
   WHERE D_{20} = EFFECTIVE SIZE (CM)
   REFERENCE: JUSTIN, HINDS, CRAEGER, ENGINEERING FOR DAMS, VOLUME 3, 1947.
2. REFERENCE: ASCE NATIONAL CONVENTION, LIQUEFACTION PROBLEMS IN GEOTECHNICAL ENGINEERING, SEPT. 27 - OCT. 1, 1976.

DIVISION	DESCRIPTION	DATE	APPROVAL
REVISIONS			

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

SANTA ANA RIVER, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM
FOUNDATION MATERIALS
DETAILED TEST RESULTS

DESIGNED BY | DRAWN BY | CHECKED BY | SUBMITTED BY | DATE APPROVED | SPEC. NO. DACW09- - - | SHEET | DISTRICT FILE NO.

PLATE A-52

SAFETY PAYS

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

LAH 86-1

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
				NP	95	17	SILTY SAND; brown, dry, medium dense.
	SM			NP	98	25	light brown.
9.0'				NP	93	16	loose to medium dense, hole caving slightly.
12.0'	SP-SM			NP	94	12	POORLY GRADED SAND WITH SILT; brown, dry, gravel to 1".
16.0'	SW-SM			NP	85	12	WELL GRADED SAND WITH SILT AND GRAVEL; brown, dry, gravel and decomposed diorite fragments to 2".
17.0'	SM			NP	78	13	SILTY SAND WITH GRAVEL; brown, dry rock fragments to 2".
19.5'	SW-SM			NP	80	9	WELL GRADED SAND WITH SILT AND GRAVEL; brown, dry, rock fragments to 5".
				NP	65	9	POORLY GRADED SAND WITH SILT AND GRAVEL; brown, dry, decomposed diorite fragments to 4", bucket breaking up rock.
25.0'	SP-SM			NP	68	8	
30.0'	SW-SM			NP	72	11	WELL GRADED SAND WITH SILT AND GRAVEL; brown, slightly damp, rock fragments to 4".
	SM			NP	75	16	SILTY SAND WITH GRAVEL; brown, slightly damp, decomposed broken rock fragments to 4"
35.0'				NP	74	13	gray-brown.
	SW-SM			NP	75	12	WELL GRADED SAND WITH SILT AND GRAVEL; gray-brown, dry, broken rock to 2", difficult drilling.
43.0'				NP	66	10	rock fragments to 5".
44.0'	SM			NP	69	14	SILTY SAND WITH GRAVEL; gray-brown, dry, rock fragments to 4", hard drilling.
	SP-SM			NP	64	9	POORLY GRADED SAND WITH SILT AND GRAVEL; gray-brown, dry, rock fragments to 4".
51.0'				NP	61	10	rock fragments to 6", hard drilling.
54.0'	SW-SM			NP	73	11	WELL GRADED SAND WITH SILT AND GRAVEL; gray-brown, dry, rock fragments to 4", easier drilling at 53'
57.0'	SM			NP	77	14	SILTY SAND WITH GRAVEL; gray-brown, damp, breaking up boulder at 56'
	SP-SM			NP	67	11	POORLY GRADED SAND WITH SILT AND GRAVEL; gray-brown, dry, rock fragments to 4".
63.0'				NP	67	11	moist, rock fragments to 6".
65.0'	SM			NP	82	15	SILTY SAND WITH GRAVEL; gray-brown, dry, 10% rock fragments to 6"
66.0'	SP-SM			NP	80	12	POORLY GRADED SAND WITH SILT AND GRAVEL; gray-brown, dry.
	SM			NP	83	20	SILTY SAND WITH GRAVEL; gray-brown, dry,
70.0'				NP	75	13	10% rock fragments to 8".
	SW-SM			NP	58	9	WELL GRADED SAND WITH SILT AND GRAVEL; gray-brown slightly damp, rock fragments to 4".
				NP	74	10	
78.5'				NP	71	11	easier drilling.
	SP-SM			NP	58	8	POORLY GRADED SAND WITH SILT AND GRAVEL; gray-brown, slightly damp, rock fragments to 4".
83.5'				NP	68	10	
86.5'	SW-SM			NP	64	10	WELL GRADED SAND WITH SILT AND GRAVEL, gray, brown, slightly damp, rock fragments to 4".
88.0'	SM			NP	77	14	SILTY SAND WITH GRAVEL, gray-brown, slightly damp, rocks to 4".
90.0'	SW-SM			NP	65	10	WELL GRADED SAND WITH SILT AND GRAVEL; caving, no progress.

LAH 86-2

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
	SM			NP	89	16	SILTY SAND, dark brown, damp, medium dense, roots, rock fragments to 5"
4.0'				NP	92	15	brown, dry.
	SW-SM			NP	96	8	WELL GRADED SAND WITH SILT, brown, dry, medium dense, broken diorite fragments to 12".
12.0'				NP	86	12	
	SM	2		NP	85	13	SILTY SAND WITH GRAVEL, brown, dry, medium dense.
20.0'		2		NP	87	14	SILTY SAND; brown, dry, hard drilling
	SP-SM			NP	72	12	POORLY GRADED SAND WITH SILT AND GRAVEL, brown, dry, dense, rock fragments to 4"
29.0'				NP	64	12	gravel to 2", hard drilling.
32.0'	SM	4		NP	68	14	SILTY SAND WITH GRAVEL, brown, dry, rock fragments to 2", hard drilling.
36.0'	SW-SM			NP	68	11	WELL GRADED SAND WITH SILT AND GRAVEL, gray-brown, dry, dense, drilling on large rocks.
				NP	81	20	SILTY SAND WITH GRAVEL, gray-brown, broken rock fragments to 2", hard drilling.
	SM			NP	75	15	very hard drilling, rock fragments to 4".
47.5'		4		NP	85	17	drilling more difficult, used new teeth and pad, very little progress, possibly on bedrock.
				NP	74	17	

LAH 86-4

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
		4		NP	78	17	SILTY SAND WITH GRAVEL, brown, barely damp, roots, gravel to 2".
				NP	79	19	dry, 5% broken rock fragments to 6".
				NP	76	24	10% rock fragments to 8", 1-12" rock inside of hole at 7'.
	SM			NP	71	18	yellow-brown, few rocks to 6".
				NP	84	16	occasional rock fragments to 4".
		3		NP	86	13	SILTY SAND, yellow-brown, dry, 4" max rocks.
				NP	77	13	SILTY SAND WITH GRAVEL, yellow-brown, dry, rock fragments to 4".
19.0'		3		NP	81	15	
				NP	82	11	WELL GRADED SAND WITH SILT AND GRAVEL,
				NP	67	9	yellow-brown, dry, 10% rock fragments
	SW-SM	2		NP	62	7	to 6".
25.0'							
26.0'	SM			NP	77	14	SILTY SAND WITH GRAVEL, yellow-brown,
				NP	63	9	dry, caving.
				NP	65	6	
	SW-SM			NP	77	11	WELL GRADED SAND WITH SILT AND GRAVEL, yellow-brown, dry, rock fragments to 6"
31.0'							
33.0'	SM	4		NP	86	18	SILTY SAND, yellow-brown, barely damp, 10% rock fragments to 8".
35.0'	SW-SM			NP	69	12	WELL GRADED SAND WITH SILT AND GRAVEL, yellow-brown, dry, large rock fell in hole, stopped drilling.

LAH

DEPTH	LOG	MC	LL	PI	-4	-200
		3		NP	69	19
	SM			NP	82	13
				NP	90	15
		4		NP	95	19
16.0'						
19.0'	SW-SM			NP	85	9
	SM			NP	82	13
22.0'						
				NP	67	8
	SW-SM	3		NP	69	8
				NP	66	8
31.0'						
	SP-SM			NP	62	7
35.0'						
		3		NP	71	10
	SW-SM			NP	62	9
		4		NP	76	9
43.0'						
44.0'	SM			NP	74	14

LAH

DEPTH	LOG	MC	LL	PI	-4	20
3.0'	SW-SM			NP	74	7
6.0'	SM	4		NP	74	13
7.0'	SW-SM			NP	80	12
10.0'	SM			NP	81	15
				NP	71	12
	SW-SM			NP	85	11
				NP	75	11
19.0'		2		NP	73	12

LAH 86-3

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
		3		NP	69	19	SILTY SAND WITH GRAVEL, brown, slightly damp, gravel and broken rock fragments to 3", hard drilling due to rocks.
	SM			NP	82	13	2" max. gravel.
				NP	90	15	SILTY SAND, brown, slightly damp, dense, gravel to 2".
		4		NP	95	19	orange-brown, easy drilling.
16.0'							
	SW-SM			NP	85	9	WELL GRADED SAND WITH SILT AND GRAVEL, orange-brown, slightly damp, broken rock fragments to 3".
19.0'							
	SM			NP	82	13	SILTY SAND WITH GRAVEL, dark brown, slightly damp, rock fragments to 4", hard drilling from 19' to 20'.
22.0'							
				NP	67	8	WELL GRADED SAND WITH SILT AND GRAVEL, brown, slightly damp, medium dense, broken rock fragments to 4".
	SW-SM	3		NP	69	8	
				NP	66	8	
31.0'							
	SP-SM			NP	62	7	POORLY GRADED SAND WITH SILT AND GRAVEL, brown, slightly damp, medium dense, rock fragments to 4".
35.0'							
		3		NP	71	10	WELL GRADED SAND WITH SILT AND GRAVEL, gold-brown, slightly damp, hard drilling due to rocks at 36'.
	SW-SM			NP	62	9	
		4		NP	76	9	brown, broken rock fragments to 4".
43.0'							
44.0'	SM			NP	74	14	SILTY SAND WITH GRAVEL, brown, damp, rock fragments to 4", caving, very little progress.

LAH 86-5

DEPTH	LOG	MC	LL	PI	-4	200	DESCRIPTION
	SW-SM			NP	74	7	WELL GRADED SAND WITH SILT AND GRAVEL, brown, damp, roots, 5% cobbles to 8".
3.0'							
	SM	4		NP	74	13	SILTY SAND WITH GRAVEL, brown, damp, 5% cobbles to 8".
6.0'							
7.0'	SW-SM			NP	80	12	WELL GRADED SAND WITH SILT AND GRAVEL, brown, dry, few cobbles to 9", caving.
	SM			NP	81	15	SILTY SAND WITH GRAVEL, brown, dry, cobbles to 5", caving.
10.0'							
				NP	71	12	WELL GRADED SAND WITH SILT AND GRAVEL, brown, dry, slow progess due to caving.
	SW-SM			NP	85	11	numerous cobbles to 9".
				NP	75	11	
19.0'		2		NP	73	12	stopped drilling due to caving.

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOILS More than half of material is larger than no. 200 sieve size.	GRAVELS More than half of coarse fraction is larger than no. 4 sieve size.	Clean gravels	GW	Well-graded gravels, gravel-sand mixtures, little or no fines.
			GP	Poorly-graded gravels, gravel-sand mixtures, little or no fines.
		Gravels with fines	GM	Silty gravels, gravel-sand-silt mixtures.
			GC	Clayey gravels, gravel-sand-clay mixtures.
	SANDS More than half of coarse fraction is smaller than no. 4 sieve size.	Clean sands	SW	Well-graded sands, gravelly sands, little or no fines.
			SP	Poorly-graded sands, gravelly sands, little or no fines.
		Sands with fines	SM	Silty sands, sand-silt mixtures.
			SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS More than half of material is smaller than no. 200 sieve size.	SILTS AND CLAYS	Low liquid limit	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts, with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays.
			OL	Organic silts and organic silty clays of low plasticity.
		High liquid limit	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			CH	Inorganic clays of high plasticity.
			OH	Organic clays of medium to high plasticity, organic silts.
Highly organic soils			Pt	Peat and other highly organic soils.

NOTES:

1. Boundary Classification: Soils possessing characteristics of two groups are designated by combinations of group symbols. For example, GW-GC, well-graded gravel-sand mixture with clay binder.
2. All sieve sizes on this chart are U. S. Standard.
3. The terms "silt" and "clay" are used respectively to distinguish materials exhibiting lower plasticity from those with higher plasticity. The minus no. 200 sieve material is silt if the liquid limit and plasticity index plot below the "A" line on the plasticity chart, and is clay if the liquid limit and plasticity index plot above the "A" line on the chart.
4. The Soil Classification System is based on the American Society for Testing and Materials (ASTM).
   a. (ASTM) D2487 Standard Test Method for Classification of Soils for Engineering Purposes.
   b. (ASTM) D2488 Standard Recommended Practice for Description of Soils (Visual Manual Procedure).

LEGEND

MC	FIELD MOISTURE CONTENT IN PERCENT OF DRY WEIGHT.
LL	LIQUID LIMIT.
PI	PLASTICITY INDEX (LIQUID LIMIT MINUS PLASTIC LIMIT).
NP	NONPLASTIC
-4	PERCENT OF MATERIAL BY WEIGHT PASSING NO. 4 SIEVE.
-200	PERCENT OF MATERIAL BY WEIGHT PASSING NO. 200 SIEVE.

NOTES:

1. TEST HOLES WERE DRILLED IN NOVEMBER, 1986 WITH A 24-INCH DIAMETER POWER BUCKET AUGER.
2. SEE PLATE A-9 FOR LOCATION OF TEST HOLES.
3. SEE PLATE A-54 FOR LABORATORY TEST RESULTS.

SCALE 1 IN. = 5 FT.

5 0 5 10 15

DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

SYMBOL	DESCRIPTION	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:

DRAWN BY: KAA

CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

LEFT ABUTMENT

SOIL LOGS

SUBMITTED BY: DATE

APPROVED:

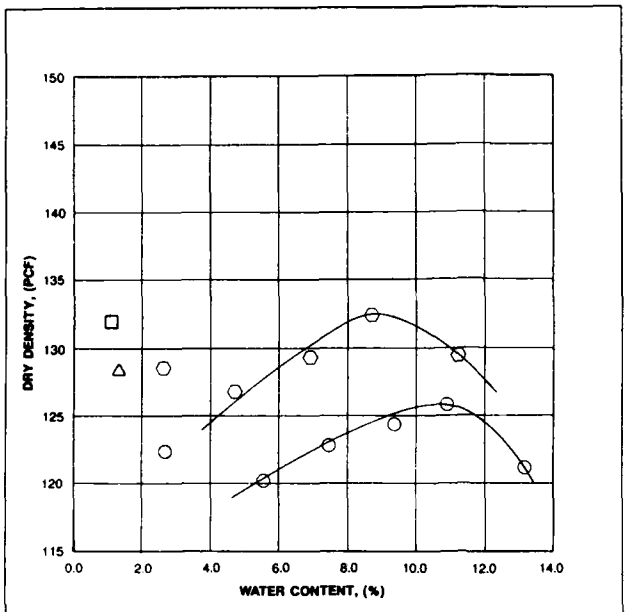
SPEC. NO. DACW09-____-____

DISTRICT FILE NO.

SHEET

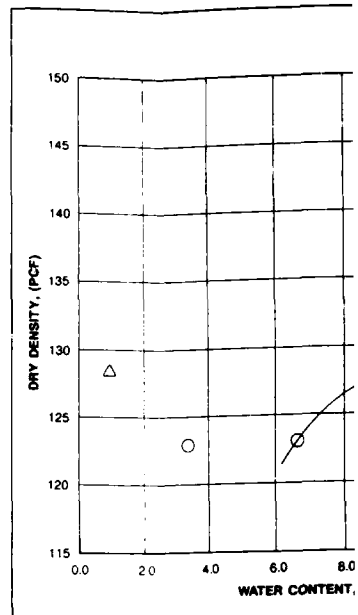
PLATE A-53

ENVIRONMENTAL
ENHANCEMENT
THRU ENGINEERING



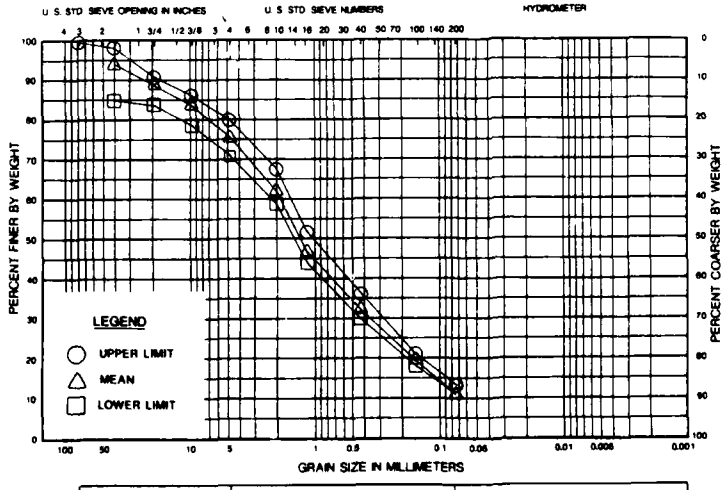
SYMBOL	HOLE/ SAMPLE NUMBER	ELEV / DEPTH (FT)	OPT MOIST CONT (%)	MAX DRY DEN (PCF)	USCS CLASS	%> No 4 SIEVE	%< No.200 SIEVE	LL	PI	Sp G	REMARKS
○	LAH-1	25.0'-26.0'	10.8	125.9	SP-SM	25	10		NP		ASTM D 698-78(C)
◯	LAH-1	81.0'-83.5'	8.8	132.3	SW-SM	32	10		NP		ASTM D 698-78(D)
△	LAH-1	25.0'-26.0'	----	----	SP-SM	25	10		NP		ASTM D 4253-83
□	LAH-1	81.0'-83.5'	----	----	SW-SM	32	10		NP		ASTM D 4253-83

COMPACTION TEST DATA



SYMBOL	HOLE/ SAMPLE NUMBER	ELEV / DEPTH (FT)	OPT MOIST CONT (%)	MAX DRY DEN (PCF)	USCS CLASS	%> No 4 VE	No SI
○	LAH-2	COMP BLEND	9.5	128.1	SM	15	
△	LAH-2	COMP BLEND	----	----	SM	15	

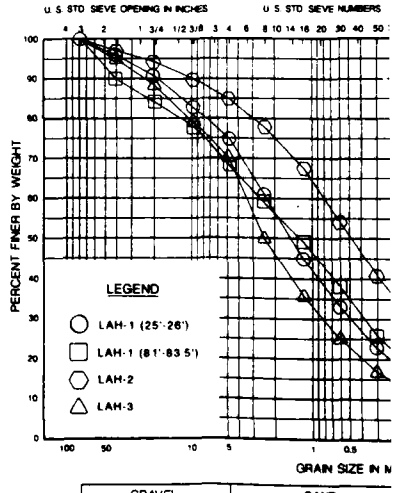
COMPACTION TEST I



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

GRADATION CURVES

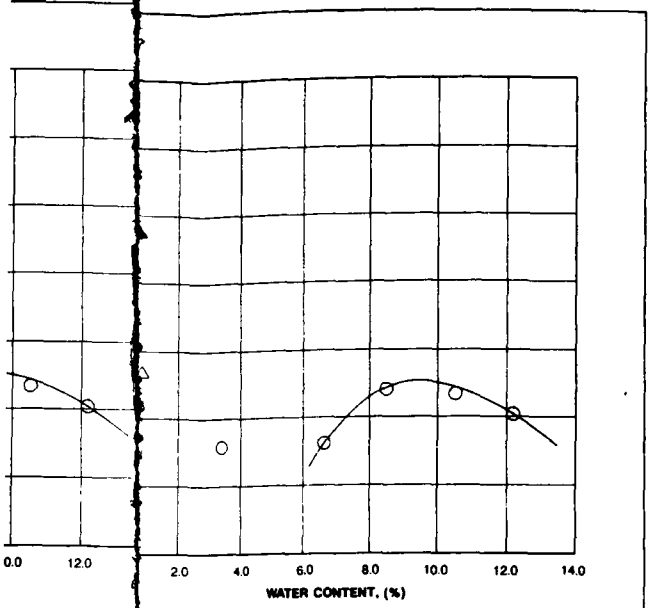
COMPOSITE BLEND



GRAVEL		SAND	
COARSE	FINE	COARSE	MEDIUM

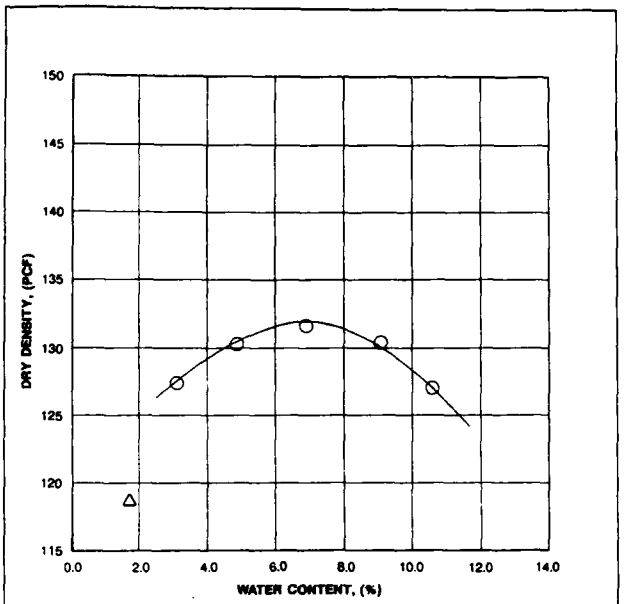
GRADATION

COMPACTION



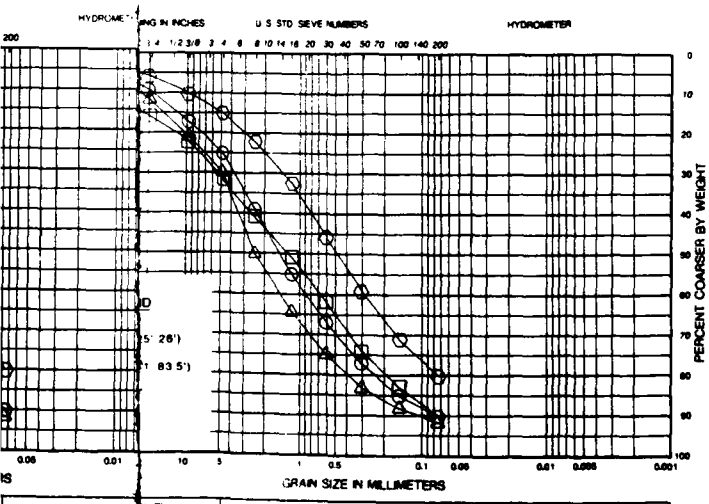
PI	Sp. G	RE	ELEV / DEPTH (FT)	OPT MOIST CONT (%)	MAX DRY DEN (PCF)	USCS CLASS	%> No 4 SIEVE	%< No 200 SIEVE	LL	PI	Sp. G.	REMARKS
NP		ASTM	COMP BLEND	9.5	128.1	SM	15	20		NP		ASTM D 698-78(C)
NP		ASTM	COMP BLEND	---	---	SM	15	20		NP		ASTM D 4253-83

COMPACTION TEST DATA



SYMBOL	HOLE/ SAMPLE NUMBER	ELEV / DEPTH (FT)	OPT MOIST CONT (%)	MAX DRY DEN (PCF)	USCS CLASS	%> No. 4 SIEVE	%< No.200 SIEVE	LL	PI	Sp G	REMARKS
○	LAH-3	COMP BLEND	6.9	131.8	SW-SM	30	8		NP		ASTM D 698-78(D)
△	LAH-3	COMP BLEND	---	---	SW-SM	30	8		NP		ASTM D 4253-83

COMPACTION TEST DATA



SILT OR CLAY	GRAVEL FINE	SAND COARSE	SAND MEDIUM	SAND FINE	SILT OR CLAY

GRADATION CURVES

COMPACTION SAMPLES

NOTES:

1. REFER TO PLATE A-9 FOR LOCATION OF TEST HOLES LAH86-1 THRU LAH86-5.
2. REFER TO PLATE A-53 FOR THE SOIL LOGS OF TEST HOLES LAH86-1 THRU LAH86-5
3. TEST METHOD ASTM D 698-78(METHOD C & D) WAS FOLLOWED WITH THE EXCEPTION THAT A MECHANICAL RAMMER HAVING A CIRCULAR CONTACT FACE TWO INCHES IN DIAMETER WAS USED WITH THE SIX INCH DIAMETER COMPACTION MOLD.

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

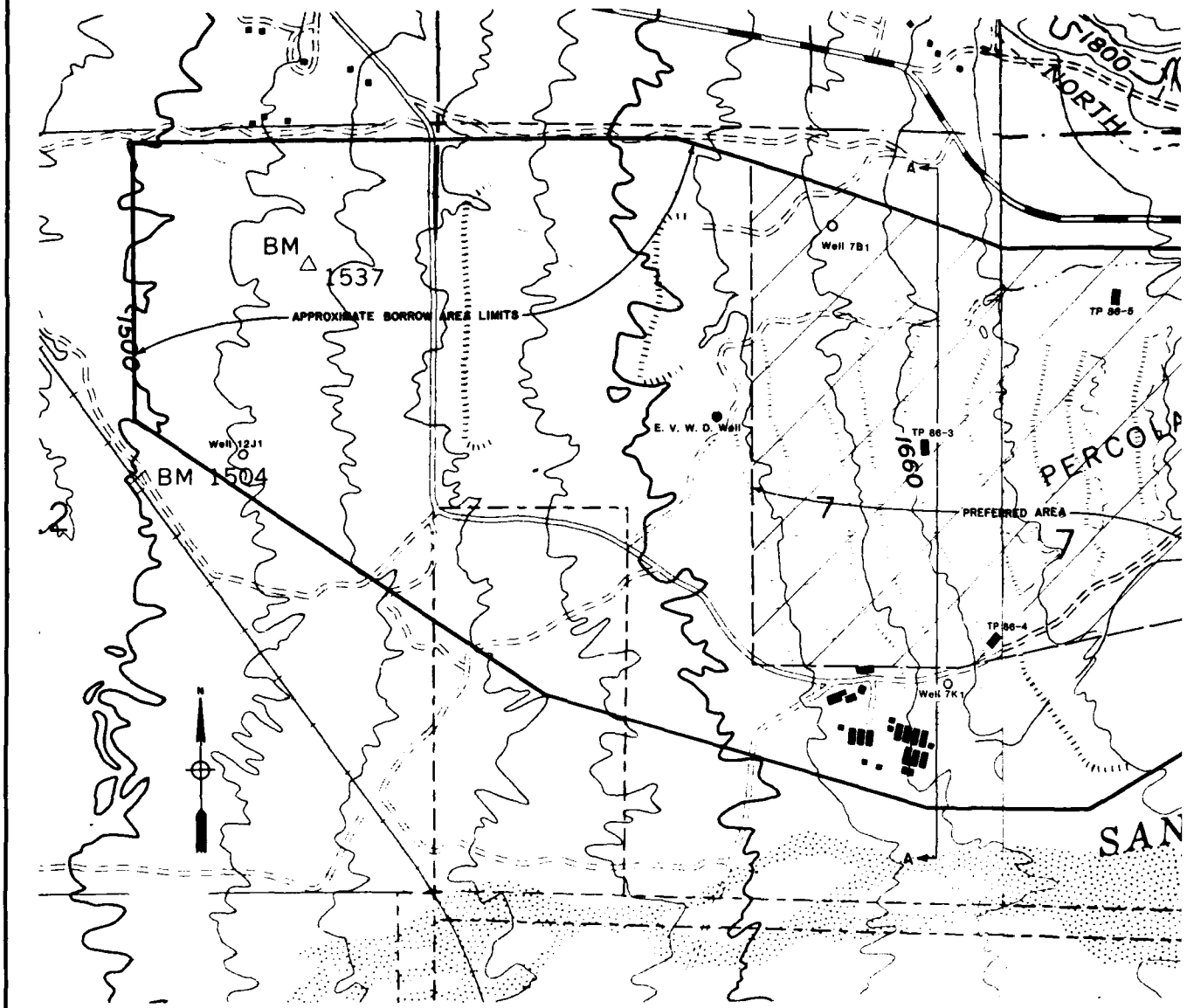
SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

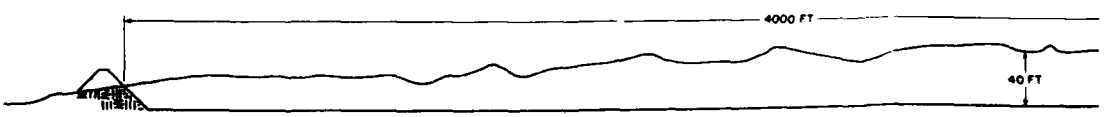
LEFT ABUTMENT
RESULTS OF LABORATORY TESTS

SUBMITTED BY:
DATE APPROVED:
SPEC. NO. DACW09- ____ B- ____
DISTRICT FILE NO.
SHEET

PLATE A-54



PLAN

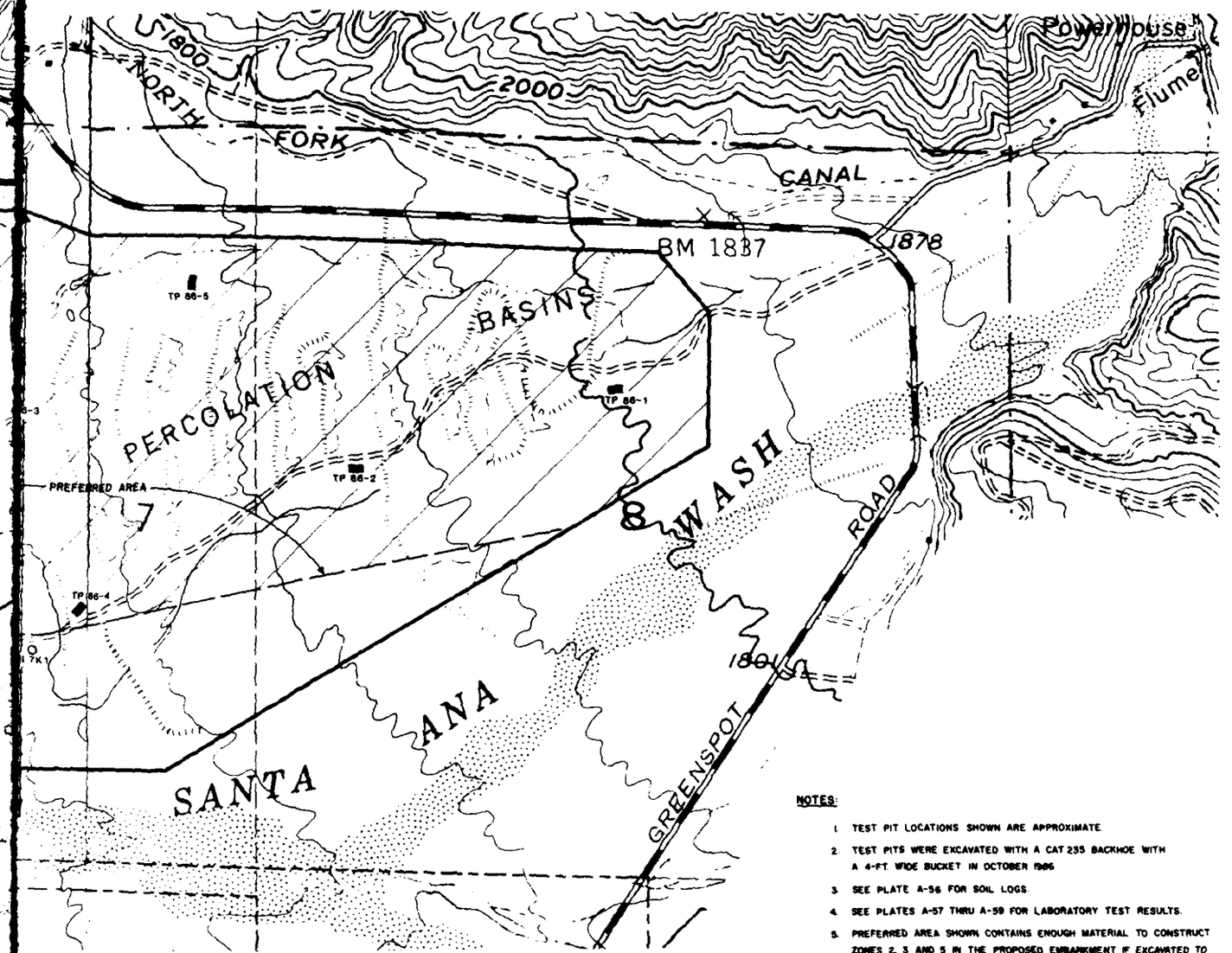


SECTION A-A

TYPICAL EXCAVATION CONFIGURATION

ENVIRONMENTAL
ENHANCEMENT
THRU ENGINEERING

ALUE ENGINEERING PAYS

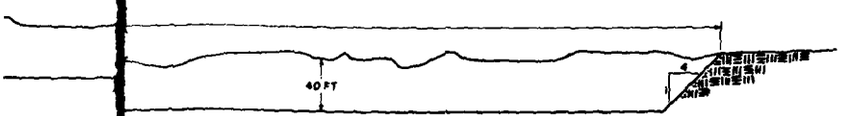


NOTES:

1. TEST PIT LOCATIONS SHOWN ARE APPROXIMATE.
2. TEST PITS WERE EXCAVATED WITH A CAT 235 BACKHOE WITH A 4-FT WIDE BUCKET IN OCTOBER 1986.
3. SEE PLATE A-96 FOR SOIL LOGS.
4. SEE PLATES A-57 THRU A-59 FOR LABORATORY TEST RESULTS.
5. PREFERRED AREA SHOWN CONTAINS ENOUGH MATERIAL TO CONSTRUCT ZONES 2, 3 AND 5 IN THE PROPOSED EMBANKMENT IF EXCAVATED TO AN AVERAGE DEPTH OF 40 FT AS SHOWN IN SECTION A-A.
6. SEE PLATE A-101 FOR CONSTRUCTION SEQUENCING AND PERCOLATION BASIN RECONSTRUCTION DETAILS.

SCALE: 1 IN. = 400 FT

SCALE 500 0 500 1000 FEET



A-A
ONFIGURATION

DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

SYMBOL	DESCRIPTION	DATE	APPROVAL
REVISIONS			

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

DOWNSTREAM PERVIOUS BORROW AREA
PLAN OF EXPLORATION

SUBMITTED BY: DATE APPROVED:

SPEC. NO. DACW09-____-____

DISTRICT FILE NO.

SHEET

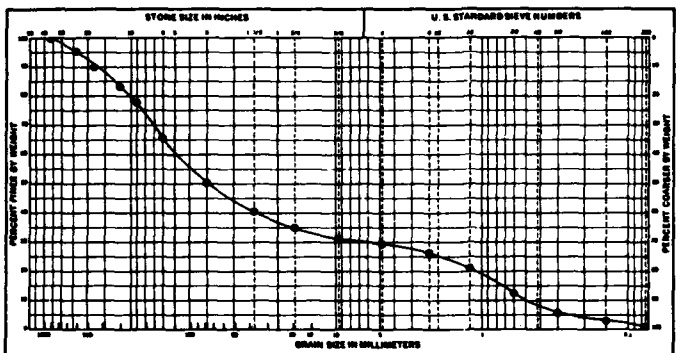
PLATE A-5

SAFETY PAYS

ENVIRONMENTAL
ENHANCEMENT
THRU ENGINEERING

TEST PIT 86-1

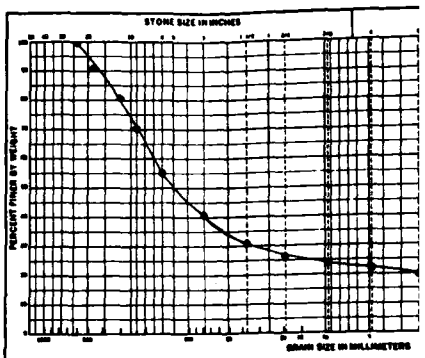
DEPTH	VISUAL LOG
24.0'	GRAVELLY SAND: 49% cobbles and boulders up to 2.5'x4'x6', 14 rocks larger than 1' minimum diameter were excavated from pit, rocks are interlocked and had to be pried out by backhoe, moist below 8' depth. 58,020 lb. sample for mass gradation shown below excavated from full depth of west wall of test pit.



MASS GRADATION

TEST PIT 86-2

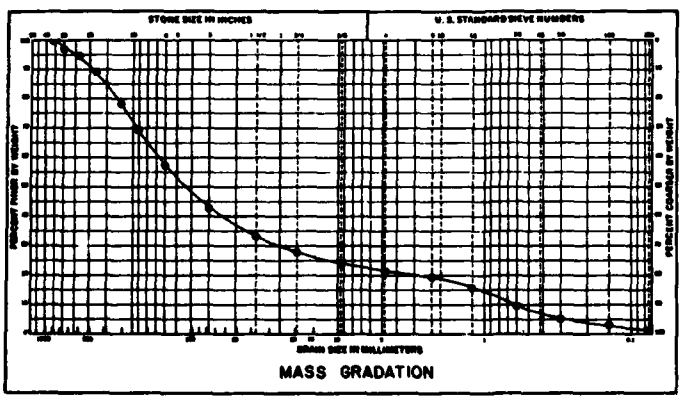
DEPTH	VISUAL LOG
24.0'	GRAVELLY SAND: 59% cobbles and boulders up to 4'x5 diameter were excavated from pit, large rocks are interlo south wall, moist below 12' depth. 38,837 lb. sample for mass gradation shown below excava test pit.



MASS GRADATION

TEST PIT 86-4

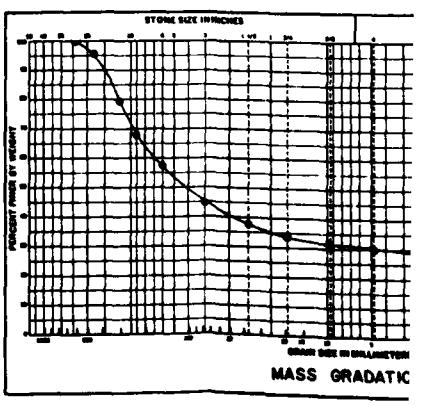
DEPTH	VISUAL LOG
18.0'	GRAVELLY SAND: 57% cobbles and boulders up to 3'x4'x6', 16 rocks larger than 1' in diameter were excavated from pit, large rocks interlocked with some broken up by backhoe during excavation, moist entire depth. 37,906 lb. sample for mass gradation shown below excavated from full depth of west wall of of test pit.



MASS GRADATION

TEST PIT 86-5

DEPTH	VISUAL LOG
18.0'	GRAVELLY SAND: 55% cobbles and boulders up to 3 diameter were excavated from pit, some open-graded g moist entire depth. 23,656 lb. sample for mass gradation shown below exc of test pit.



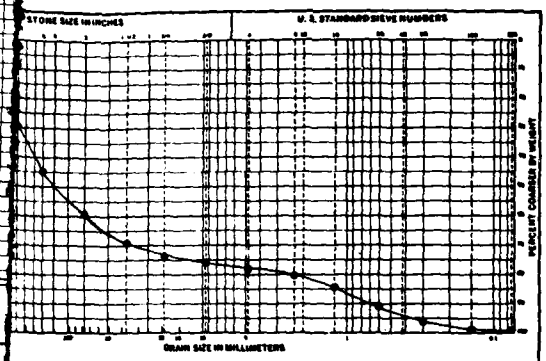
MASS GRADATIC

PAY

ALUE ENGINEERING PAYS

TEST PIT 86-2

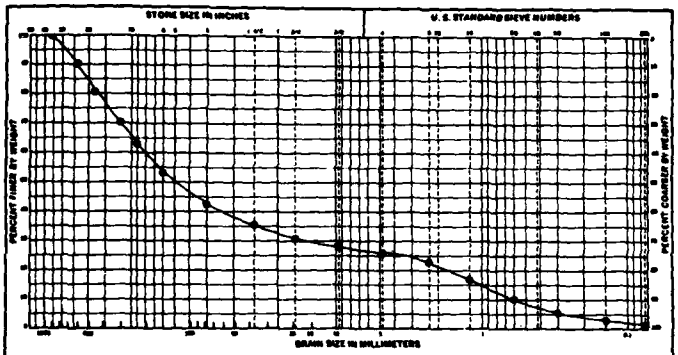
VISUAL LOG
SAND: 58% cobbles and boulders up to 4'x5'x7', 32 rocks larger than 1' in re excavated from pit, large rocks are interlocked, 2'x3' sand pocket at 12' on moist below 12' depth.
sample for mass gradation shown below excavated from full depth of west wall of



MASS GRADATION

TEST PIT 86-3

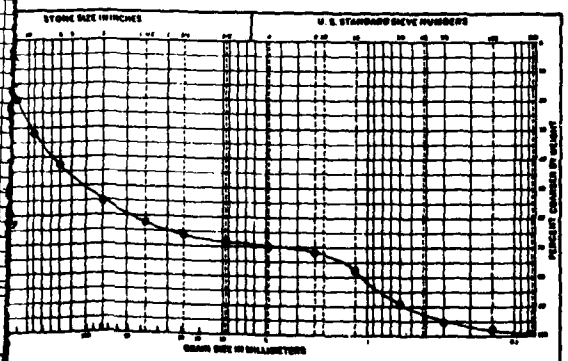
DEPTH	VISUAL LOG
	GRAVELLY SAND: 58% cobbles and boulders up to 3'x3.5'x4.5', 14 rocks larger than 1' in diameter were excavated from pit, moist entire depth.
	40,792 lb. sample for mass gradation shown below excavated from full depth of north wall of test pit.
20.0'	



MASS GRADATION

TEST PIT 86-5

VISUAL LOG
LLY SAND: 55% cobbles and boulders up to 3'x4'x6', 14 rocks larger than 1' in r were excavated from pit, some open-graded gravel layers visible in pit walls, ntire depth.
lb. sample for mass gradation shown below excavated from full depth of north wall pit.



MASS GRADATION

NOTES:

1. TEST PITS WERE EXCAVATED WITH A CAT 235 BACKHOE WITH A 4'-WIDE BUCKET IN OCTOBER 1986.
2. SEE PLATE A-55 FOR LOCATION OF TEST PITS.
3. SEE PLATES A-57 AND A-58 FOR LABORATORY TEST RESULTS.

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

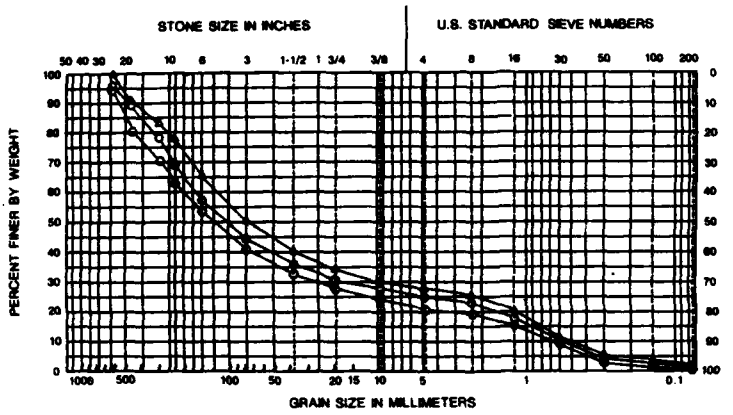
SEVEN OAKS DAM

DOWNSTREAM PERVIOUS BORROW AREA
SOIL LOGS AND GRADATIONS

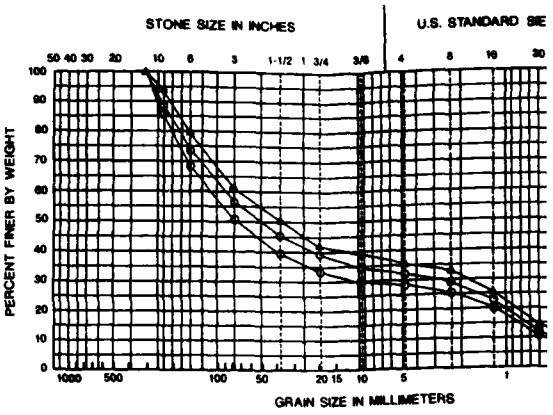
SUBMITTED BY:
DATE APPROVED:
CHIEF, BRANCH
SPEC. NO. DACW09-____-____
DISTRICT FILE NO.
SHEET

SAFETY PAYS

PLATE A-5

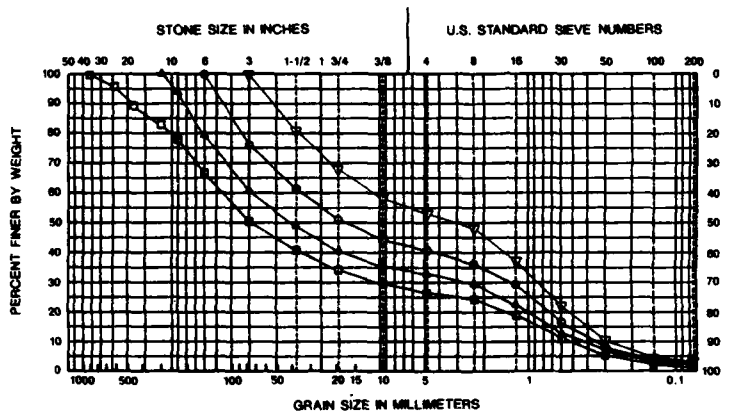


MASS GRADATION

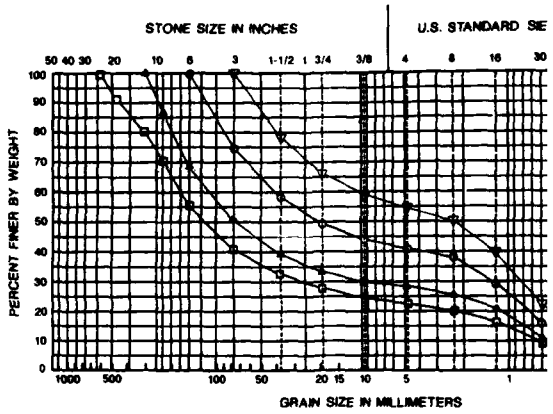


-12 INCH GRADATION

BLEND OF TEST PITS TP86-1 TO TP8

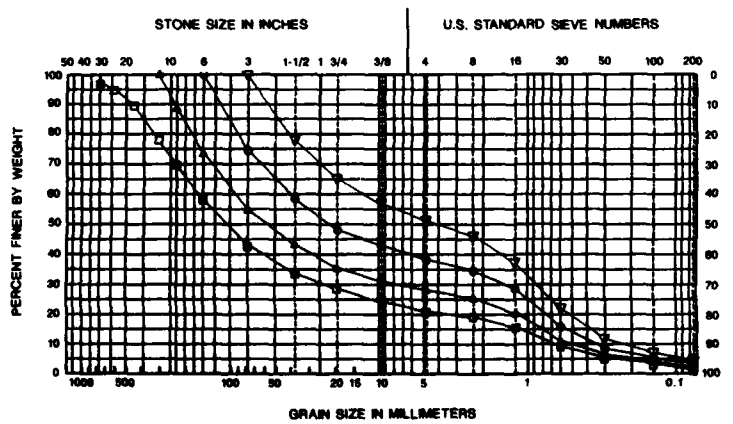


TP86-1

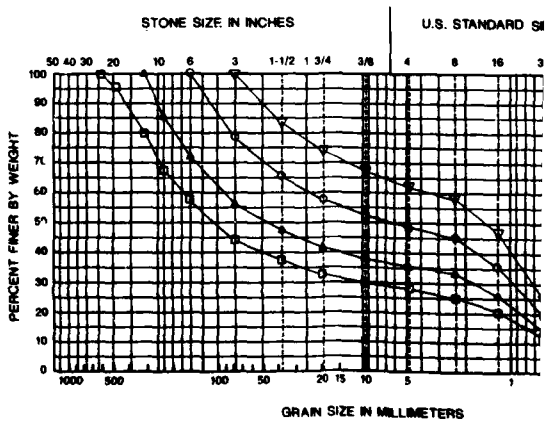


TP86-2

TEST PITS TP86-1 TO TP86-5

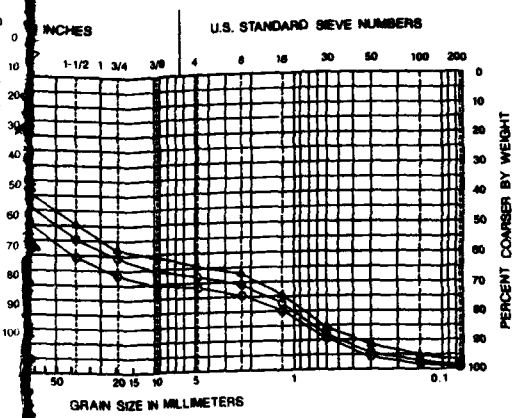


TP86-4

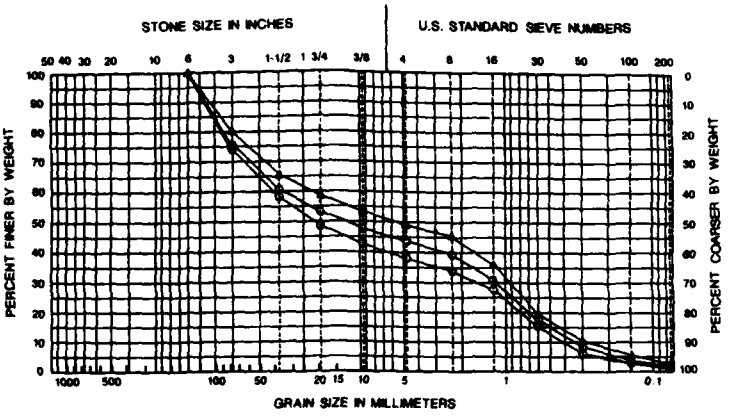


TP86-5

ENVIRONMENTAL
ENHANCEMENT
THRU ENGINEERING



-12 INCH GRADATION

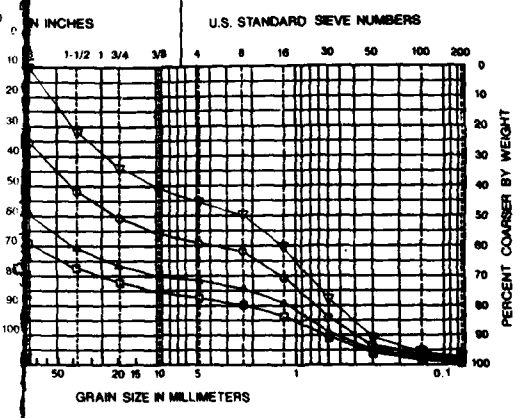


-6 INCH GRADATION

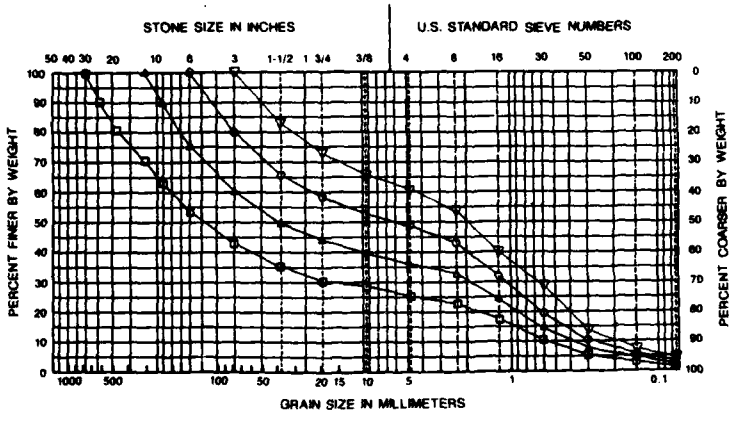
LEGEND

- △ UPPER LIMIT
- ○ MEAN
- ○ LOWER LIMIT

OF TEST PITS TP86-1 TO TP86-5



TP86-2

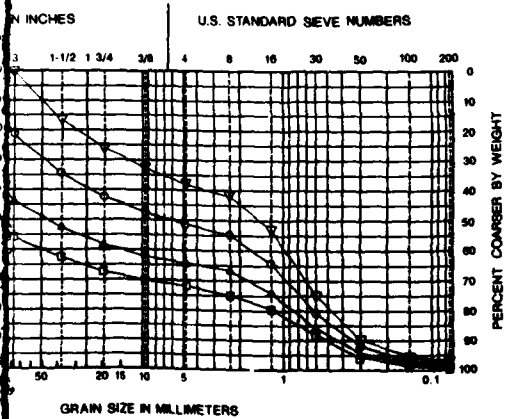


TP86-3

LEGEND

- □ MASS GRADATION
- △ -12" GRADATION
- ○ -6" GRADATION
- ▽ -3" GRADATION

ST PITS TP86-1 TO TP86-5



TP86-5

NOTES:

1. REFER TO PLATE A-56 FOR SOIL LOGS
2. REFER TO PLATE A-55 FOR LOCATION OF TEST PITS TP86-1 THRU TP86-5

SYMBOL	DESCRIPTION	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY: J.T.B.
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

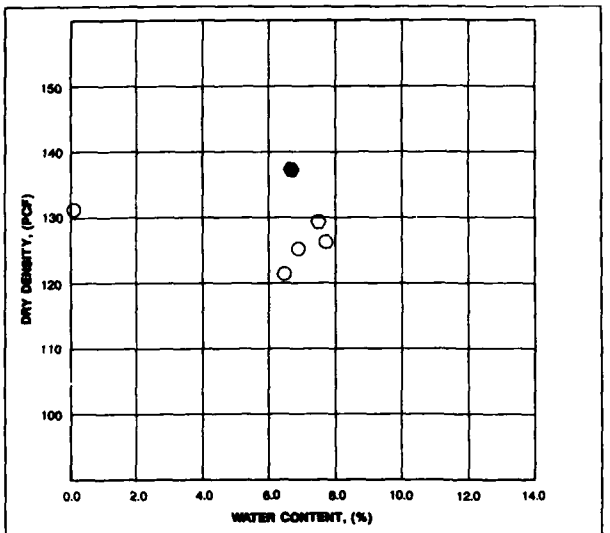
SEVEN OAKS DAM
DOWNSTREAM PERVIOUS BORROW AREA
GRADATION CURVES

SUBMITTED BY: DATE
APPROVED:
CHIEF, ENGINEERING DIVISION

SPEC. NO. DACW09-____-____
DISTRICT FILE NO.

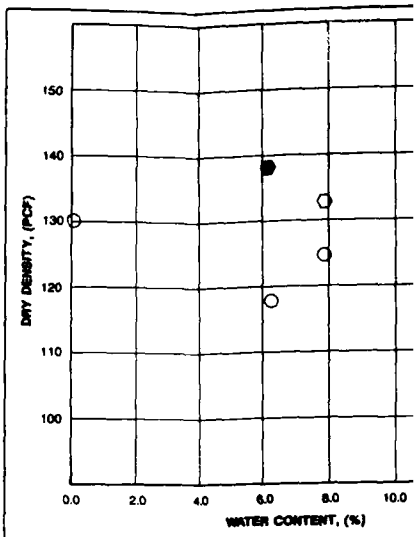
SHEET

PLATE A-51



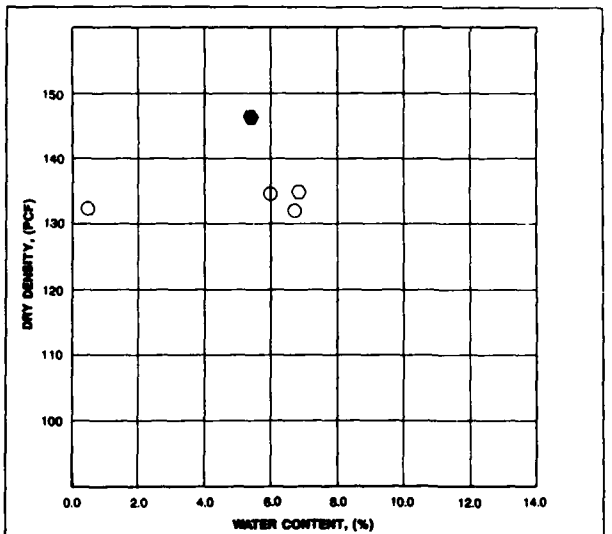
SYMBOL	HOLE/ SAMPLE NUMBER	ELEV./ DEPTH (FT)	OPT. MOIST. CONT (%)	MAX. DRY DEN. (PCF)	USCS CLASS.	MECHANICAL ANALYSIS GR	SA	FI	LL	PI	Sp. G.	REMARKS
○	TP86-1	N/A	AS SHOWN	AS SHOWN	SP	47	51	2		NP		REFER TO NOTE 1
⬡	TP86-1	N/A	7.5	129.3	SP	43	55	2		NP		REFER TO NOTE 2
⬢	TP86-1	N/A	6.7	137.2	GP	59	39	2		NP		REFER TO NOTE 3

TP86-1



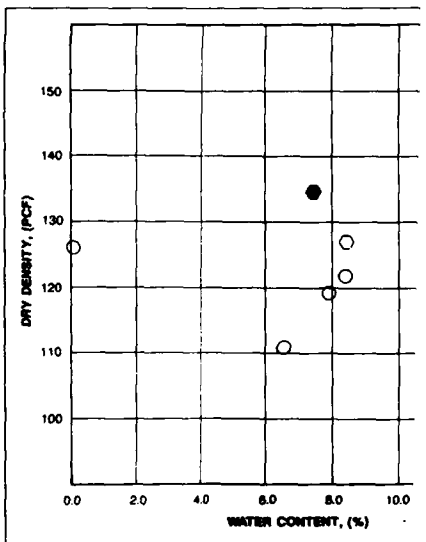
SYMBOL	HOLE/ SAMPLE NUMBER	ELEV./ DEPTH (FT)	OPT MOIST CONT (%)	MAX DRY DEN. (PCF)	USCS CLASS	MECHANICAL ANALYSIS GR	SA	FI	LL
○	TP86-2	N/A	AS SHOWN	AS SHOWN	SP	45	53	2	
⬡	TP86-2	N/A	7.9	132.8	SP	46	52	2	
⬢	TP86-2	N/A	6.1	138.4	GP	59	40	1	

TP86-2



SYMBOL	HOLE/ SAMPLE NUMBER	ELEV./ DEPTH (FT)	OPT. MOIST. CONT. (%)	MAX. DRY DEN. (PCF)	USCS CLASS.	MECHANICAL ANALYSIS GR	SA	FI	LL	PI	Sp. G.	REMARKS
○	TP86-4	N/A	AS SHOWN	AS SHOWN	SP	49	47	4		NP		REFER TO NOTE 1
⬡	TP86-4	N/A	6.9	134.7	SP	48	48	4		NP		REFER TO NOTE 2
⬢	TP86-4	N/A	5.4	148.3	GP	62	35	3		NP		REFER TO NOTE 3

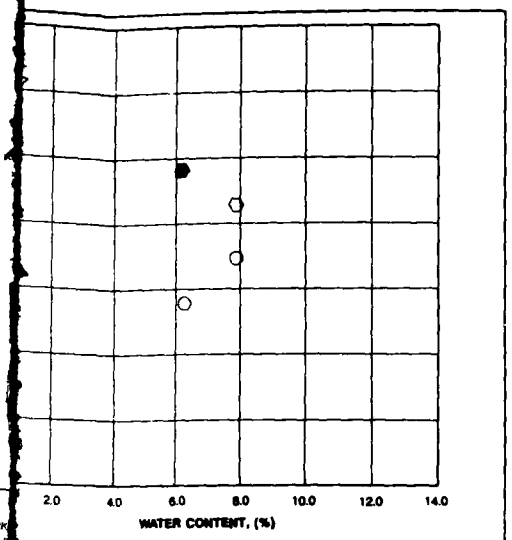
TP86-4



SYMBOL	HOLE/ SAMPLE NUMBER	ELEV./ DEPTH (FT)	OPT MOIST CONT (%)	MAX DRY DEN (PCF)	USCS CLASS.	MECHANICAL ANALYSIS GR	SA	FI	LL
○	TP86-5	N/A	AS SHOWN	AS SHOWN	SP	38	60	2	
⬡	TP86-5	N/A	8.4	128.9	SP	33	65	2	
⬢	TP86-5	N/A	7.5	134.6	GP	51	47	2	

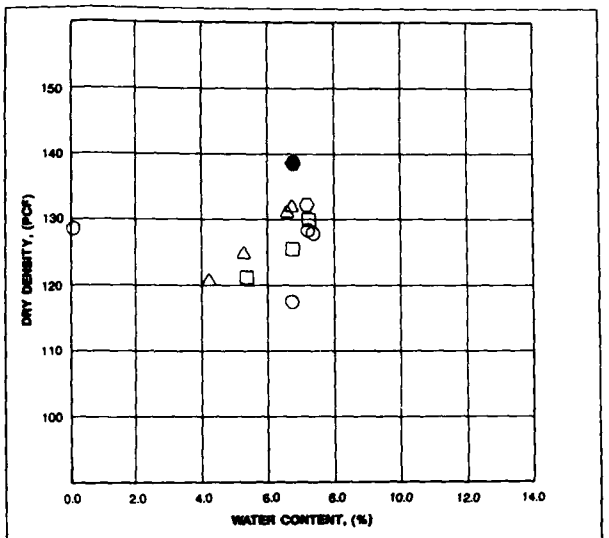
TP86-5

VALUE ENGINEERING PAYS



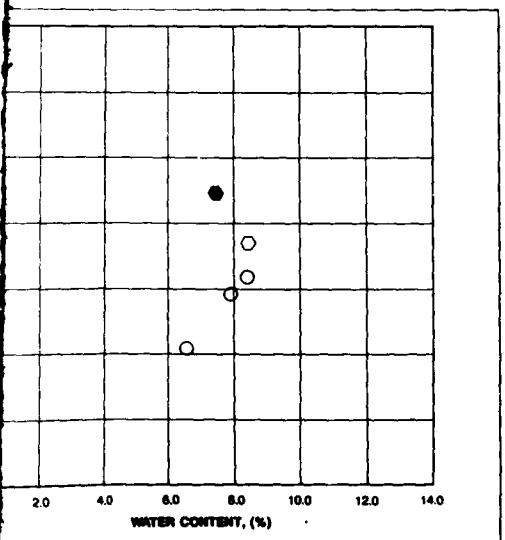
ELEV./ DEPTH (FT)	OPT MOIST CONT (%)	MAX DRY DEN (PCF)	USCS CLASS	MECHANICAL ANALYSIS GR	SA	FI	LL	PI	Sp. G.	REMARKS
N/A	AS SHOWN	AS SHOWN	SP	45	53	2		NP		REFER TO NOTE 1
N/A	7.9	132.8	SP	46	52	2		NP		REFER TO NOTE 2
N/A	6.1	138.4	GP	59	40	1		NP		REFER TO NOTE 3

TP86-2



SYMBOL	HOLE/ SAMPLE NUMBER	ELEV./ DEPTH (FT)	OPT. MOIST. CONT. (%)	MAX. DRY DEN. (PCF)	USCS CLASS.	MECHANICAL ANALYSIS GR	SA	FI	LL	PI	Sp. G.	REMARKS
○	TP86-3	N/A	AS SHOWN	AS SHOWN	SP	49	57	4		NP		REFER TO NOTE 1
⬡	TP86-3	N/A	7.2	132.3	SP	49	57	4		NP		REFER TO NOTE 2
⬢	TP86-3	N/A	6.8	138.8	GP	51	46	3		NP		REFER TO NOTE 3
□	TP86-3	N/A	6.8	132.0	SP	39	56	5		NP	2.89	REFER TO NOTE 4
△	TP86-3	N/A	7.2	130.0	SP	38	57	5		NP	2.83	REFER TO NOTE 4

TP86-3



ELEV./ DEPTH (FT)	OPT. MOIST. CONT (%)	MAX DRY DEN (PCF)	USCS CLASS.	MECHANICAL ANALYSIS GR	SA	FI	LL	PI	Sp. G.	REMARKS
N/A	AS SHOWN	AS SHOWN	SP	38	60	2		NP		REFER TO NOTE 1
N/A	8.4	126.9	SP	33	65	2		NP		REFER TO NOTE 2
N/A	7.5	134.6	GP	51	47	2		NP		REFER TO NOTE 3

TP86-5

NOTES:

1. SAMPLES WITH A MAXIMUM PARTICLE SIZE OF 3 INCHES WERE TESTED USING ASTM METHOD D 4253-83.
2. SAMPLES WITH A MAXIMUM PARTICLE SIZE OF 9 INCHES WERE TESTED IN A 27-INCH DIAMETER VIBRATORY MOLD IN ACCORDANCE WITH CALIFORNIA STATE DEPARTMENT OF WATER RESOURCES LARGE SCALE, VIBRATORY MAXIMUM DENSITY TEST METHOD.
3. SAMPLES WITH A MAXIMUM PARTICLE SIZE OF 6 INCHES WERE TESTED IN A 27-INCH DIAMETER VIBRATORY MOLD IN ACCORDANCE WITH CALIFORNIA STATE DEPARTMENT OF WATER RESOURCES LARGE SCALE, VIBRATORY MAXIMUM DENSITY TEST METHOD.
4. SAMPLES WITH A MAXIMUM PARTICLE SIZE OF 3 INCHES WERE TESTED USING ASTM METHOD D 4253-83 AT THE SOUTH PACIFIC DIVISION LABORATORY.
5. REFER TO PLATE A-56 FOR SOIL LOGS AND MASS GRADATIONS OF TP86-1 THRU TP86-5.
6. REFER TO PLATE A-57 FOR APPROXIMATE GRADATIONS OF THE TESTED SAMPLES.

SYMBOL	DESCRIPTION	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

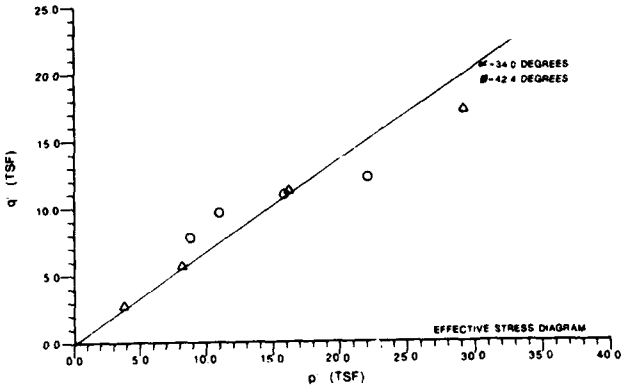
SEVEN OAKS DAM

DOWNSTREAM PERVIOUS BORROW AREA
MAXIMUM DENSITY TESTS

SUBMITTED BY:
DATE APPROVED:
SPEC. NO. DACW09-___-8-___
DISTRICT FILE NO.
SHEET

PLATE A-58

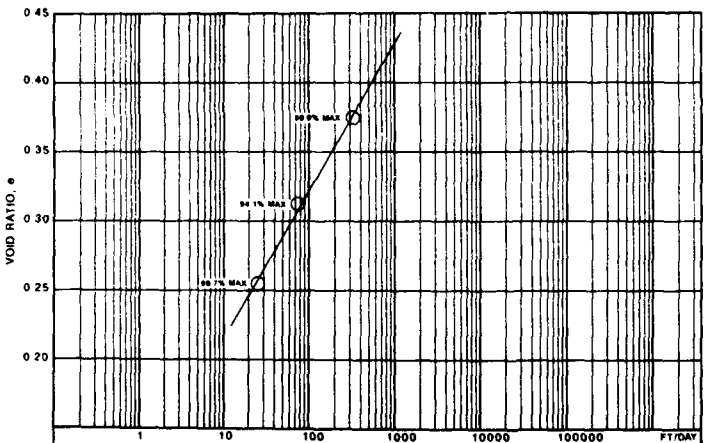
SAFETY PAYS



p'-q' DIAGRAM

TP86-3

ENVIRONMENTAL
ENHANCEMENT
THRU ENGINEERING



SYMBOL	HOLE/ SAMPLE NUMBER	ELEV / DEPTH (FT)	USCS CLASS	MOIST CONT (%)	DRY DENS (PCF)	MAX PART (IN.)	MECHANICAL ANALYSIS GR	SA	FI	ATTERBERG LIMITS LL	PI	REMARKS
○	TP86-3	COMP BLEND	GRAVELLY SAND (SP)	OPT 8.8	MAX 132	3	58	36	5		NP	ASTM D 4253-83

NOTES

PERMEABILITY TEST DATA

PA

LUE ENGINEERING PAYS

NOTES:

1 REFER TO PLATE A-55 FOR LOCATION OF TEST PITS

2 REFER TO PLATE A-56 FOR SOIL LOGS AND GRADATIONS

3 REFER TO PLATE A-58 FOR MOISTURE-DENSITY RELATIONSHIPS

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:

DRAWN BY:

CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

DOWNSTREAM PERVIOUS BORROW AREA
RESULTS OF DETAILED LABORATORY TESTS

SUBMITTED BY:

DATE APPROVED:

SPEC. NO. DACW09- ____ -B- ____

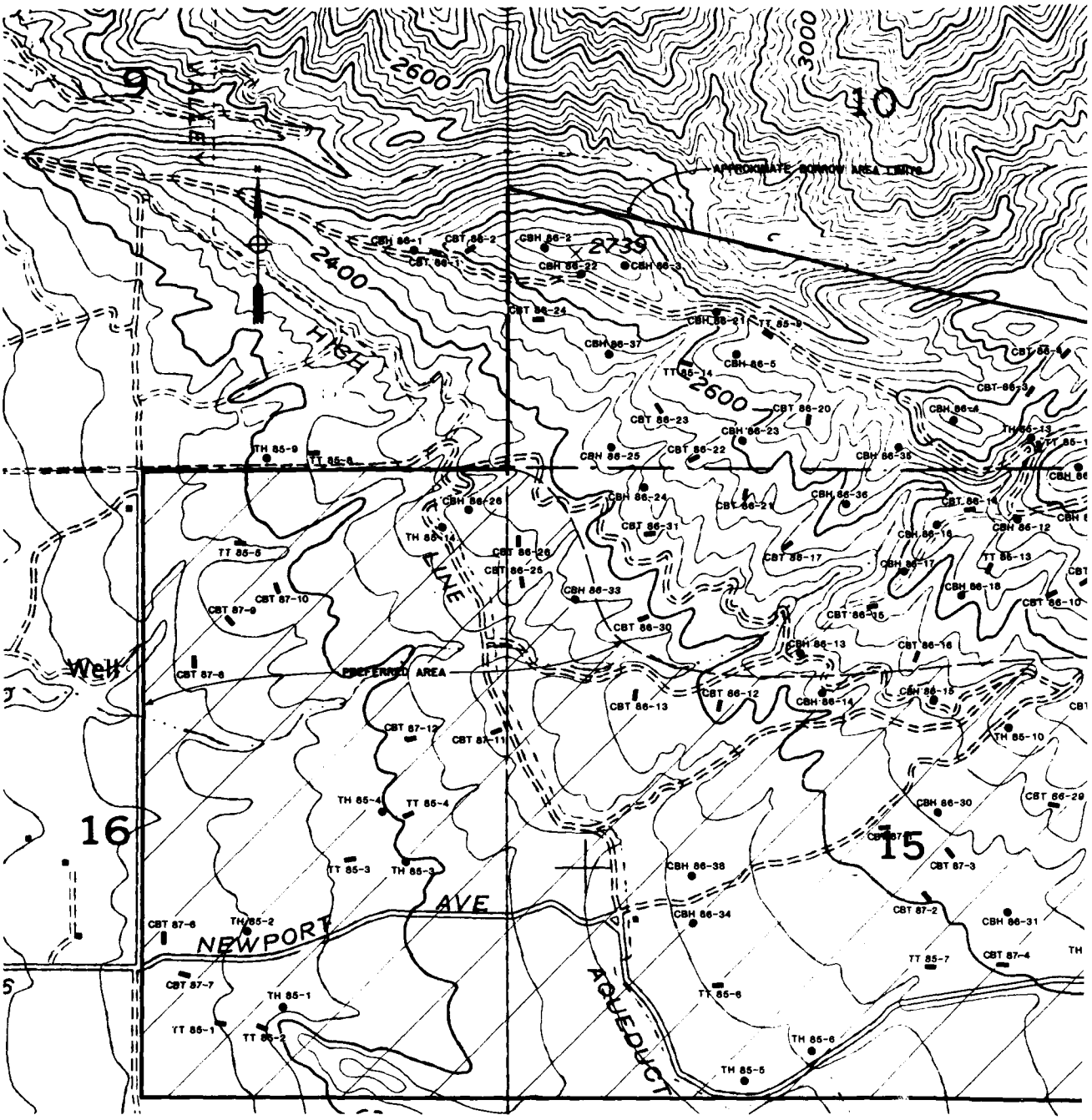
DISTRICT FILE NO.

SHEET

DATE _____ SERIES _____

SAFETY PAYS

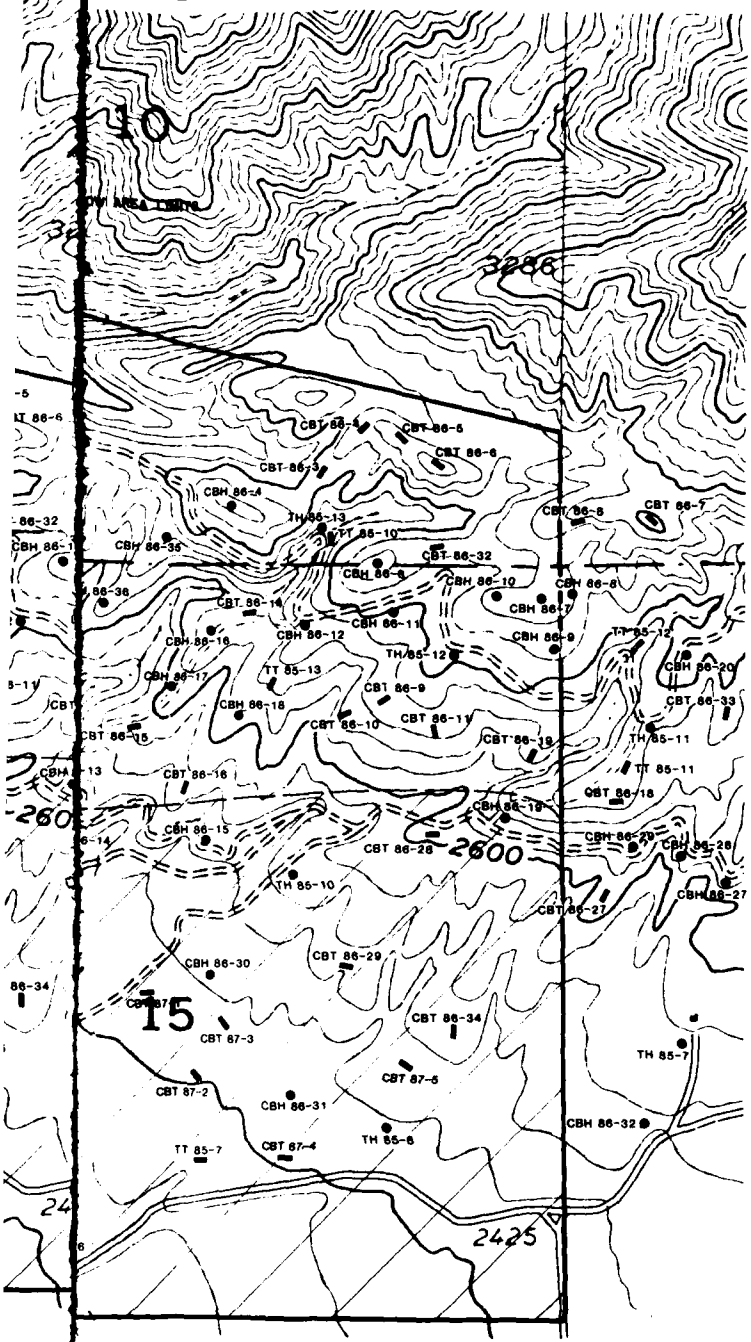
PLATE A-59



ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

P

VALUE ENGINEERING PAYS



NOTES:

1. TEST HOLE AND TEST TRENCH LOCATIONS SHOWN ARE APPROXIMATE.
2. TEST HOLES TH 85-1 THROUGH TH 85-14 WERE DRILLED WITH A 24" BUCKET AUGER IN JUNE 1985.
3. TEST TRENCHES TT 85-1 THROUGH TT 85-14 WERE EXCAVATED WITH A BACKHOE IN JUNE 1985.
4. TEST HOLES CBH 86-1 THROUGH CBH 86-38 WERE DRILLED WITH A 24" BUCKET AUGER IN DECEMBER 1986.
5. TEST TRENCHES CBT 86-1 THROUGH CBT 86-34 WERE EXCAVATED WITH A BACKHOE IN DECEMBER 1986.
6. SEE PLATES A-62 THROUGH A-73 FOR SOIL LOGS.
7. SEE PLATES A-74 THROUGH A-79 FOR LABORATORY TEST RESULTS.
8. PREFERRED AREA SHOWN CONTAINS APPROXIMATELY TWICE THE AMOUNT OF MATERIAL THAT IS REQUIRED TO CONSTRUCT THE IMPERVIOUS CORE OF THE PROPOSED EMBANKMENT.
9. TEST TRENCHES CBT 87-1 THROUGH CBT 87-12 WERE EXCAVATED WITH A BACKHOE IN DECEMBER 1987.

SCALE: 1 IN. = 400 FT.

SCALE 500 0 500 1000 FEET

DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

IMPERVIOUS BORROW AREA
PLAN OF EXPLORATION

SUBMITTED BY: DATE APPROVED:

CHIEF, ENGRG DIV

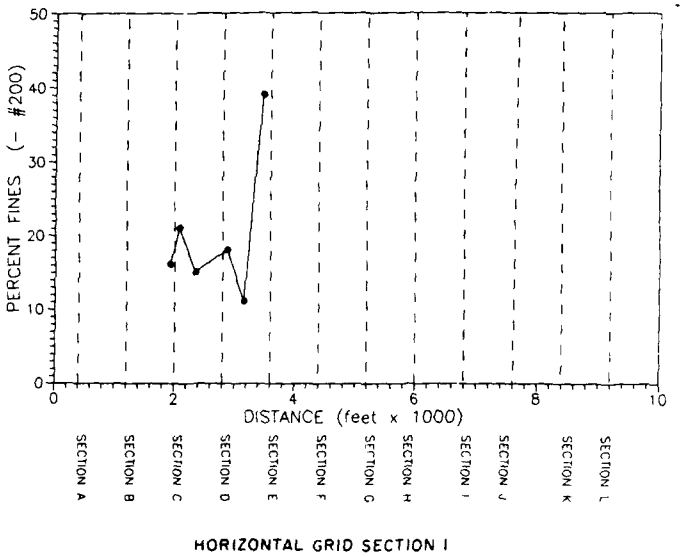
SPEC. NO. DACW 09- ____ B- ____

DISTRICT FILE NO.

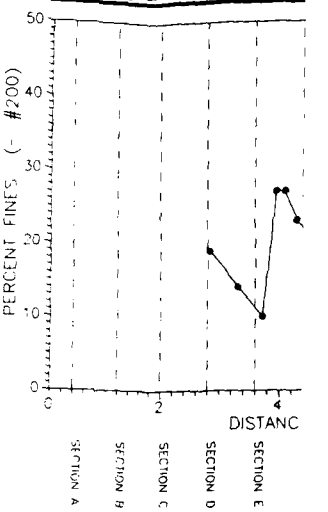
SHEET

SAFETY PAYS

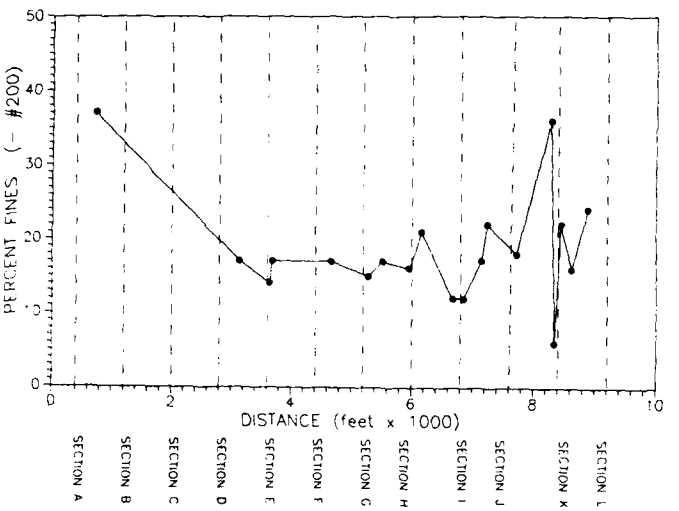
PLATE A-60



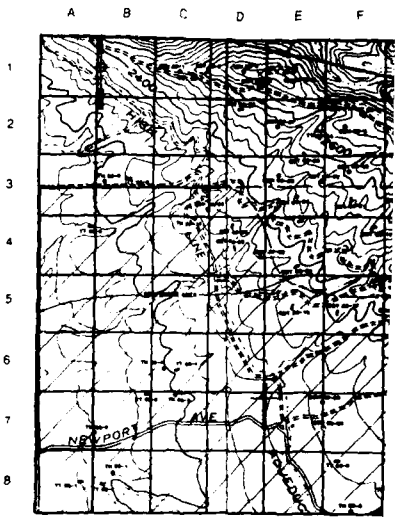
HORIZONTAL GRID SECTION I



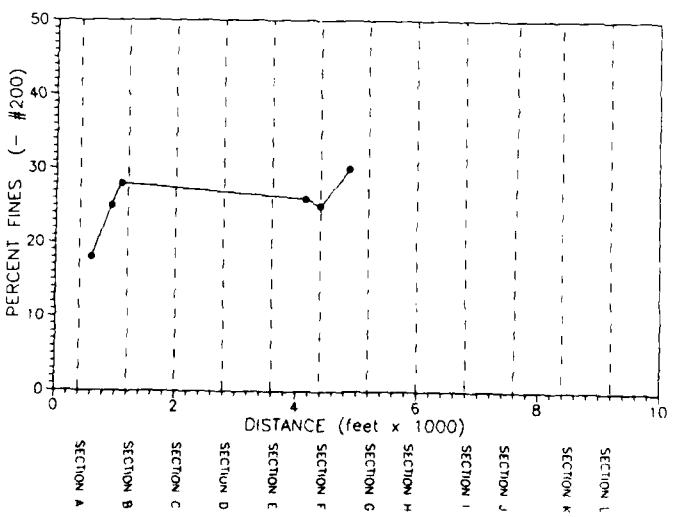
HORIZONTAL (



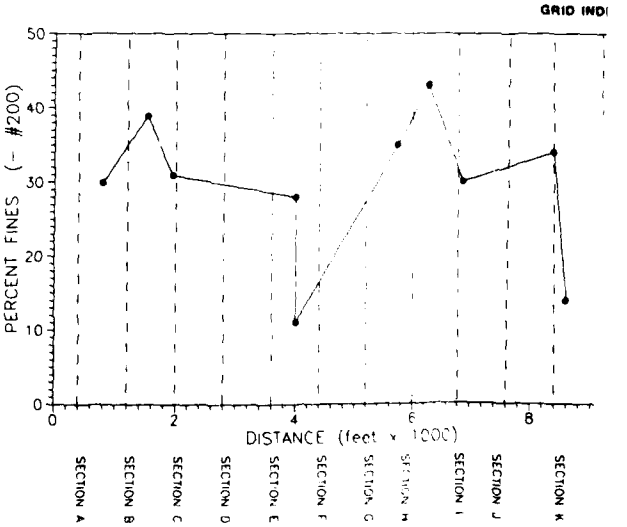
HORIZONTAL GRID SECTION 4



GRID INDI



HORIZONTAL GRID SECTION 6

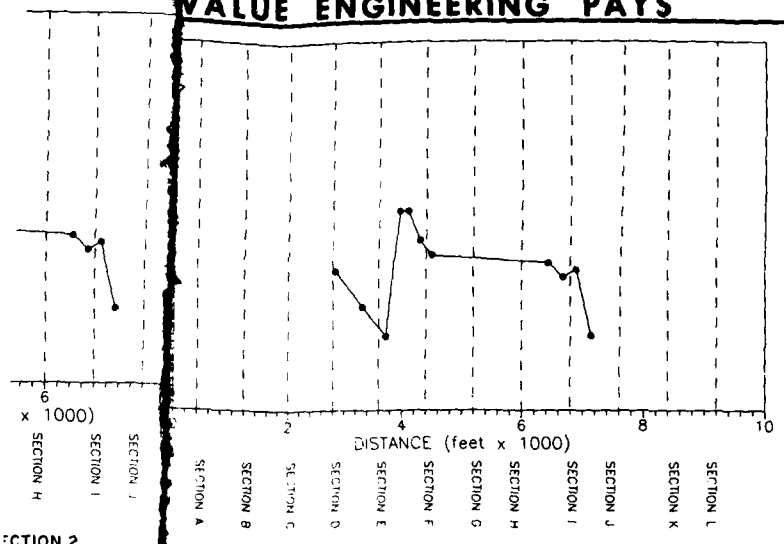


HORIZONTAL GRID SECTION 7

ENVIRONMENTAL
ENHANCEMENT
THRU ENGINEERING

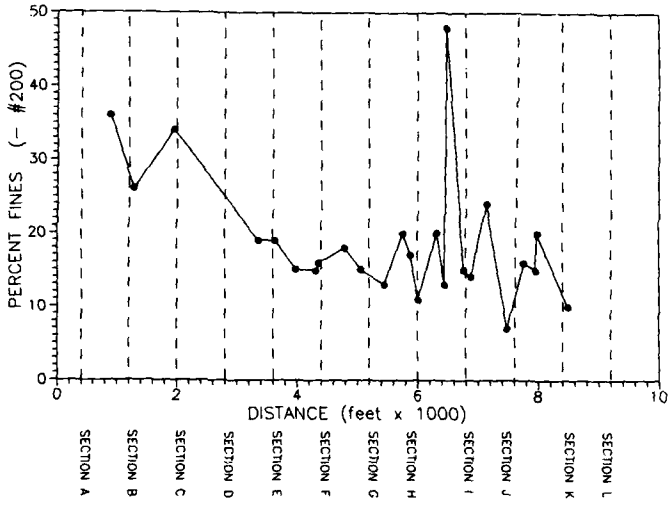
NG PAY

VALUE ENGINEERING PAYS

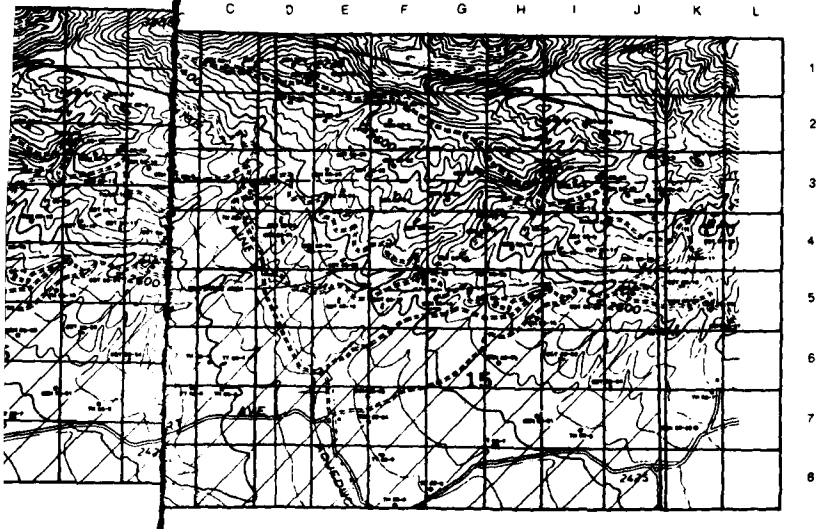


ECTION 2

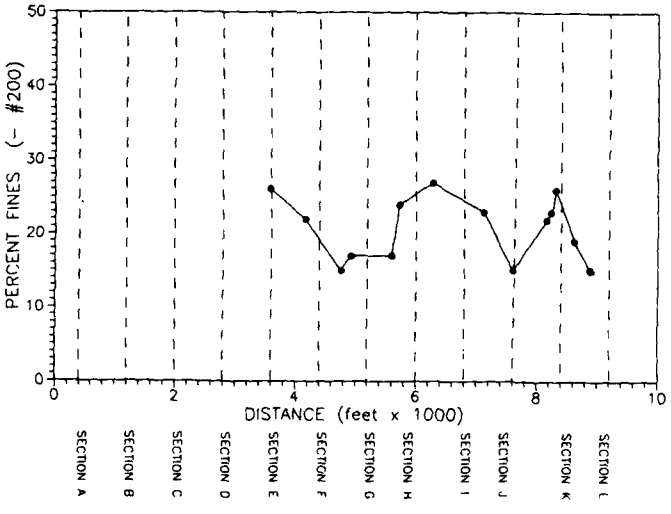
HORIZONTAL GRID SECTION 2



HORIZONTAL GRID SECTION 3



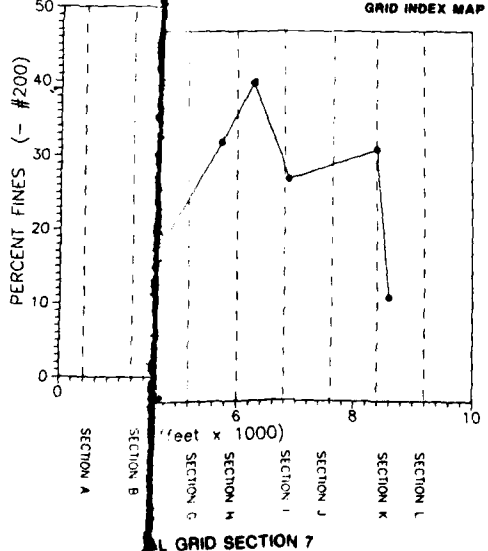
GRID INDEX MAP



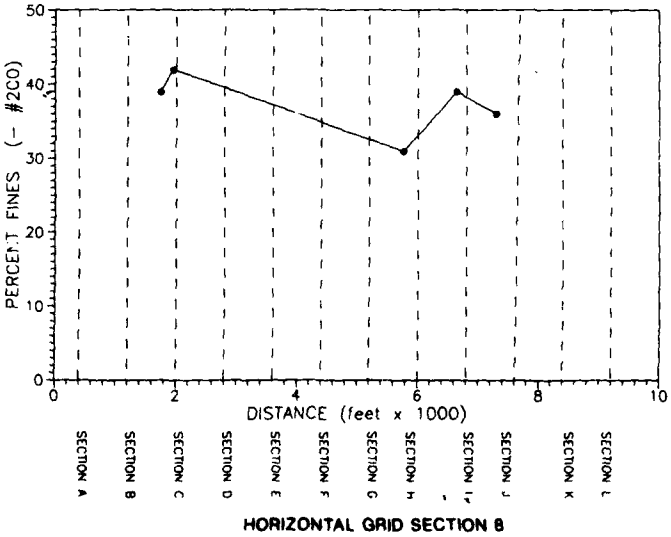
HORIZONTAL GRID SECTION 5

NOTES:

1. DATA POINTS REPRESENT THE "UN-WEIGHTED" AVERAGE OF THE PERCENT PASSING THE NUMBER 200 STANDARD SIEVE OF THE SAMPLES OCCURRING WITHIN THE TOP FIFTEEN FEET OF A SPECIFIC TEST LOCATION.
2. DATA FROM TEST TRENCHES CBT87-1 TO CBT87-12 ARE NOT INCLUDED IN THE FIGURES.
3. REFER TO PLATE A-60 FOR LOCATION OF TEST HOLES.
4. REFER TO PLATES A-62 THRU A-72 FOR SOIL LOGS.



L GRID SECTION 7



HORIZONTAL GRID SECTION 8

DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

SYMBOL	DESCRIPTION	DATE

REVISIONS

U. S. ARMY ENGINEER DIST. LOS ANGELES CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

IMPERVIOUS BORROW AREA
FINES DISTRIBUTION

SUBMITTED BY:
DATE APPROVED:
CHIEF, BRANCH
SPEC. NO. DACW 09- - -
DISTRICT FILE NO.

AYS

SAFETY PAYS

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

TH85-1

DEPTH	LOG	LL	PI	-4	-200	DESCRIPTION
	SM	19	3	95	23	SILTY SAND; brown to dark brown, moist, few gravel to 1", slight cohesion.
			NP	97	26	dark brown, 10% cobbles to 9", some cementation, breaks with finger pressure.
5.5'			NP	95	23	dense, compressible, small amount of white chalky material.
8.0'	SC	23	8	98	23	CLAYEY SAND; dark brown, moist, dense, few gravel to 3".
	SM	26	6	92	29	SILTY SAND; dark brown, moist, dense, few gravel to 3".
		23	5	99	46	few gravel to 1", some cohesion.
		28	6	79	24	GRAVELLY SILTY SAND; brown, gravel to 3", no cohesion.
19.0'		25	5	99	46	SILTY SAND; slight cohesion, 10% cobbles to 6".
	SW-SM		NP	69	6	GRAVELLY SAND/SILTY GRAVELLY SAND; brown, moist, no cohesion, decomposed granite, 10% cobbles to 6".
23.0'			NP	73	11	slight cohesion, gravel to 3".
25.0'	SP-SM		NP	59	7	no cohesion small amount of clay binder, gravel to 3".
	SW-SM		NP	65	7	30% cobbles to 9".
30.0'			NP	79	11	20% cobbles to 9".

TH85-2

DEPTH	LOG	LL	PI	-4	-200	D
		32	13	92	41	CLAYEY SAND; l
	SC	27	11	96	40	few gravel to
5.5'		31	13	96	33	more cementati
8.0'	SM	25	3	98	32	SILTY SAND; dr
		24	5	97	28	SILTY SAND/CLA
	SC-SM	22	6	90	21	few gravel to
13.0'		25	3	93	23	SILTY SAND; l
	SM		NP	93	20	few gravel to
18.0'			NP	91	11	SAND SILTY SA
	SW-SM					10% cobbles to
24.0'						large boulders

TH85-4

DEPTH	LOG	LL	PI	-4	-200	DESCRIPTION
1.0'	SC-SM	24	7	95	41	SILTY SAND/CLAYEY SAND; brown, moist, few gravel to 3".
		33	9	97	48	
	SM	29	6	98	36	SILTY SAND; dark brown, dry, cemented chunks, clayey binder, occasional gravel to 2".
5.5'						
9.0'	SC-SM	23	5	98	47	SILTY SAND/CLAYEY SAND; dark brown, moist, clayey binder, some cohesion, occasional gravel to 1½", some cemented clods.
	SM	25	3	98	38	SILTY SAND; dark brown, few gravel to 1".
11.0'						
12.5'	SC-SM	25	5	97	35	SILTY SAND/CLAYEY SAND; dark brown, dense, cohesive, few gravel to 2".
		26	4	96	29	SILTY SAND; dark brown, few gravel to 2".
		28	1	97	42	light brown, moist, occasional gravel to 3".
	SM	28	1	95	26	brown, fine to medium grained sand.
			NP	94	21	
			NP	93	16	brown to red-brown, medium to coarse grained sand, few gravel to 2". cobbles to 9".
25.5'		24	1	97	21	brown, few gravel to 3", no cobbles. cobbles and boulders.

TH85-5

DEPTH	LOG	LL	PI	-4	-200	DES
	SM	31	8	98	54	SILTY SAND, bro dense, slight co grained sand.
5.0'						
	SW-SM	23	4	75	12	GRAVELLY SAND/S
7.5'	SC-		NP	78	12	damp, fine to m to 4"
9.0'		23	5	71	13	GRAVELLY SILTY
10.0'			NP	57	11	brown, damp, fin
	SM					GRAVELLY SAND/S
	GP-GM		NP	54	9	damp, dense, gra SANDY GRAVEL/SI
14.5'						fine to medium
18.0'	SP-SM		NP	62	9	GRAVELLY SAND/S
	SW-SM		NP	73	8	gravel to 3", co
19.5'						

TH85-7

DEPTH	LOG	LL	PI	-4	-200	DESCRIPTION
						SILTY SAND; light grey-brown, dry, loose, fine grained sand.
			NP	94	15	light brown, damp, medium to fine grained sand.
			NP	91	14	slight caving started at 3.5'.
			NP	92	16	
	SM		NP	94	14	
			NP	93	13	medium dense, caving stopped at 10'.
			NP	95	16	
20.0'			NP	95	16	
22.0'	SP-SM		NP	92	12	SAND/SILTY SAND; brown, increase in moisture, damp, medium to fine grained sand.
26.0'	SM			97	18	SILTY SAND; brown, damp, medium to fine grained sand.

NOTES

1. SE
2.
3. TE
DI

NG PA
ALUE ENGINEERING PAYS

TH85-2

LOG	LL	PI	-4	-200	DESCRIPTION
SC	32	13	92	41	CLAYEY SAND; light brown, dry, dense, cemented, few gravel to 2", some silt.
	27	11	96	40	more cementation.
	31	13	96	33	
SM	25	3	98	32	SILTY SAND; dry, slight cohesion.
SC-SM	24	5	97	28	SILTY SAND/CLAYEY SAND; dark brown, slight cohesion,
	22	6	90	21	few gravel to 1".
SM	25	3	93	23	SILTY SAND; light brown, moist, slight cohesion, few gravel to 1".
		NP	93	20	
SW-SM		NP	91	11	SAND SILTY SAND; light brown, moist, no cohesion, 10% cobbles to 6".
					large boulders.

TH85-3

DEPTH	LOG	LL	PI	-4	-200	DESCRIPTION
		23	2	99	39	SILTY SAND; brown, moist, occasional gravel to 2".
		32	8	81	28	GRAVELLY SILTY SAND; dark brown, moist, gravel to 1½.
		28	3	97	27	
		24	2	95	37	SILTY SAND; medium to fine grained sand, decomposed granite.
	SM	25	2	97	30	dense, few cobbles to 6".
		23	1	93	29	
		23	1	98	26	20% cobbles to 6".
		26	2	98	23	very dense, occasional gravel to 3".
21.0'			NP	88	15	coarser grained sand.
24.0'	SW-SM		NP	80	10	GRAVELLY SAND/SILTY GRAVELLY SAND; dark brown, moist, coarse grained sand, gravel to 3".
			NP	92	24	SILTY SAND; brown, moist, coarse grained sand is decomposed granite, few gravel to 3".
28.0'	SM		NP	90	20	red, slight cohesion.
32.0'	SP-SM		NP	66	10	GRAVELLY SAND/SILTY GRAVELLY SAND; dark brown, moist, gravel to 3".
			NP	92	16	SILTY SAND; light brown, moist, medium grained sand, few gravel to 2".
			NP	100	33	brown, fine grained sand, occasional gravel to 1".
	SM		NP	98	21	dark brown, medium to fine grained sand.
			NP	97	20	brown, medium grained sand, occasional gravel to 2".
44.0'			NP	98	23	

TH85-5

LOG	LL	PI	-4	-200	DESCRIPTION
SM	31	8	98	54	SILTY SAND, brown to dark brown, dry to damp, dense, slight cohesive binder, fine to medium grained sand.
SW-SM	23	4	75	12	GRAVELLY SAND/SILTY GRAVELLY SAND, brown, damp, fine to medium grained sand, cobbles to 4".
SC-SM		NP	78	12	
SC-SM	23	5	71	13	GRAVELLY SILTY SAND/GRAVELLY CLAYEY SAND; brown, damp, fine grained gravel, gravel to 3".
SM		NP	57	11	
GP-GM		NP	54	9	GRAVELLY SAND/SILTY GRAVELLY SAND; brown, damp, dense, gravel to 3", few cobbles.
SP-SM		NP	62	9	SANDY GRAVEL/SILTY SANDY GRAVEL, brown, damp, fine to medium grained sand.
SW-SM		NP	73	8	GRAVELLY SAND/SILTY/GRAVELLY SAND, brown, damp, gravel to 3", occasional cobbles to 4".

TH85-6

DEPTH	LOG	LL	PI	-4	-200	DESCRIPTION
0.5'						
	SC-SM	24	5	95	33	SILTY SAND; light brown, dry, loose, fine grained sand.
3.5'		25	6	98	39	SILTY SAND/CLAYEY SAND; light brown to brown, dry to damp, medium dense, medium to fine grained sand.
5.5'	SC	29	10	98	37	
7.0'	SM	23	2	76	17	brown to dark brown, dense, medium to fine grained sand, cohesive fines.
7.5'	SP-SM		NP	74	10	CLAYEY SAND; brown, increase in moistures, damp, medium dense, fine grained sand.
						GRAVELLY SILTY SAND; brown, damp, dense, coarse and fine grained sand, fine grained gravel, gravel to 3", one 4" cobble.
						GRAVELLY SAND/SILTY GRAVELLY SAND; greyish brown, very dense, fine grained sand, gradation may be due to grinding and pulverizing of larger rocky material by the bucket.

NOTES

1. SEE PLATE 60 FOR LOCATION OF TEST HOLES.
2.
3. TEST HOLES WERE DRILLED USING A 24" BUCKET AUGER IN JUNE 1985.

SCALE 1 IN. = 5 FT.

SCALE 5 0 5 10 15 FEET

SYMBOL	DESCRIPTIONS	DATE	APPROV
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

IMPERVIOUS BORROW AREA
SOIL LOGS
TH85-1 THRU TH85-7

SUBMITTED BY: DATE APPROVED: SPEC. NO. DACW 09-___-B-___ DISTRICT FILE NO.

PLATE A

SAFETY PAYS

ENVIRONMENTAL
ENHANCEMENT
THRU ENGINEERING

TH85-8

DEPTH	LOG	LL	PI	-4	-200	DESCRIPTION
	SC	27	9	99	40	CLAYEY SAND; light brown, baraly damp, medium dense, fine grained sand
		27	10	100	37	dark brown, damp, fine to medium grained sand, cohesive.
8.0'						
	SM	24	2	92	18	SILTY SAND; brown, damp, medium dense, fine to medium grained sand, non-cohesive.
		24	2	93	23	
			NP	97	21	
			NP	100	25	
			NP	99	24	
			NP	95	18	
			NP	97	20	
26.0'			NP	98	24	slightly cohesive.

TH85-9

DEPTH	LOG	LL	PI	-4	-200	
2.0'	CL-ML	25	5	99	51	SANDY SILT
4.0'	SC	29	9	96	49	CLAYEY SAN fine to m
6.0'	SM	31	7	95	41	cohesive.
8.0'	SC	30	9	97	35	SILTY SAND
						CLAYEY SAN
		24	1	95	29	SILTY SAND fine to m
		22	1	96	26	cobble to
		23	2	97	38	barely coh
	SM					dense, medi
		29	5	97	27	
			NP	93	13	fine to m
			NP	96	18	
			NP	93	16	
30.0'			NP	91	15	medium den

TH85-11

DEPTH	LOG	LL	PI	-4	-200	DESCRIPTION
	SC	30	9	100	34	CLAYEY SAND; light brown, dry, medium dense, well cemented fine grained sand.
4.0'						
	SM	23	1	97	21	SILTY SAND, light brown, dry, medium dense.
			NP	94	16	
			NP	96	21	light brown
			NP	95	13	
15.0'			NP	95	16	brown.
	SC-SM	23	4	96	26	SILTY SAND/CLAYEY SAND; brown, dry, medium dense, fine to medium grained sand, few fine grained gravel, slight cohesion.
20.0'		26	4	89	26	dense, dark brown, fine grained sand.
	SM		NP	95	19	SILTY SAND; brown, medium dense, no cohesion.
23.0'			NP	93	13	dry to damp, dense.
	SW-SM		NP	89	12	SAND/SILTY SAND; dark brown, damp, dense, coarse to medium grained sand, few fine grained gravel.
26.0'	SM		NP	84	12	GRAVELLY SAND/SILTY GRAVELLY SAND, brown, fine grained gravel.
						SILTY SAND; brown, damp, dense, some whitish silt from pulverized decomposed rock, refusal at 26'.

TH85-12

DEPTH	LOG	LL	PI	-4	-200	
2.0'	SC-SM	26	6	94	20	SILTY SAN dense, sli
4.0'	SM		NP	87	19	SILTY SAN cemented,
8.0'	SC-SM	20	5	90	22	SILTY SAN dense, few
			NP	97	18	SILTY SAN grained s
	SM	32	8	89	27	few fine g
13.5'	SW-SM					SAND/SILTY
14.5'			NP	86	10	gravel to
15.5'	SM		NP	90	26	drilling
						SILTY SAN thin lens dense at

TH85-14

DEPTH	LOG	LL	PI	-4	-200	DESCRIPTION
	SM	30	7	99	36	SILTY SAND, brown, dry, dense, slight cohesion.
3.0'		31	7	95	28	medium dense.
	SC-SM	23	5	97	24	SILTY SAND/CLAYEY SAND; brown, damp, medium dense.
5.0'						
	SM	23	3	100	31	brown, damp, medium dense, fine grained sand.
8.0'						
	SC-SM	25	5	99	35	SILTY SAND/CLAYEY SAND; brown, damp, dense, clods of dark grey sand, cohesive.
14.0'		25	4	97	31	medium, slight cohesion.
16.0'	SM	22	3	97	26	SILTY SAND; brown, damp, medium dense, slight cohesion.
17.0'	SC-SM	23	4	96	26	SILTY SAND CLAYEY SAND; brown, damp, medium dense, cohesive.
	SM	21	0	93	14	SILTY SAND; brown with specks of white, damp, medium dense to dense, no cohesion.
20.0'						
21.0'	SP-SM	32	0	84	11	GRAVELLY SAND/SILTY GRAVELLY SAND; brown, damp, dense, gravel to 1".
25.0'	SW-SM	22	1	88	12	SAND/SILTY SAND, medium dense to dense.
26.0'	SM	22	1	92	13	SILTY SAND; brown, damp, medium dense.
	SC-SM	25	7	98	34	SILTY SAND/CLAYEY SAND; brown, damp, loose, cohesive.
29.0'		23	6	99	38	darker brown, increase in moisture, damp, dense.
	SM	20	0	92	20	SILTY SAND; brown, damp, medium dense, no cohesion, decomposed granite.
32.0'		22	2	95	15	
	SW-SM	20	3	86	12	SAND/SILTY SAND; brown, damp, medium dense.
35.0'						
37.0'	SM	22	1	99	29	SILTY SAND; brown with white specks, damp, medium dense.
38.0'	SW-SM	21	0	95	14	brown.
39.0'	SM	21	1	95	11	SAND/SILTY SAND; brown, damp, medium dense.
		22	1	97	27	SILTY SAND; brown, damp, medium dense, trace of decomposed granite.
		23	4	98	29	
	SC-SM	24	5	97	31	SILTY SAND/CLAYEY SAND; brown with white, damp, medium dense, cohesive, decomposed granite.
		23	4	99	38	
45.0'						
	SM	23	3	91	24	SILTY SAND, brown, damp, medium dense, decomposed granite,
47.5'		20	0	92	19	refusal at 47.5'.

PAYS

VALUE ENGINEERING PAYS

TH85-9

DEPTH	LOG	LL	PI	-4	-200	DESCRIPTION
2.0'	CL-ML	25	5	99	51	SANDY SILTY CLAY; light brown, dry, medium dense, fine grained sand.
4.0'	SC	29	9	96	49	CLAYEY SAND; dark brown, damp, dense, fine to medium grained sand, slightly cohesive.
6.0'	SM	31	7	95	41	SILTY SAND; brown, damp, dense, fine to medium grained sand, barely cohesive.
8.0'	SC	30	9	97	35	CLAYEY SAND; brown, damp, dense, fine to coarse grained sand.
	SM	24	1	95	29	SILTY SAND; brown, damp, medium dense, fine to medium grained sand, occasional cobble to 4".
		22	1	96	26	barely cohesive, no cobbles.
		23	2	97	38	dense, medium to coarse grained sand.
		29	5	97	27	
			NP	93	13	fine to medium grained sand.
			NP	96	18	
			NP	93	16	
30.0'			NP	91	15	medium dense, few fine grained gravel to 3/4".

TH85-10

DEPTH	LOG	LL	PI	-4	-200	DESCRIPTION
	SM	23	3	97	34	SILTY SAND; light brown to brown, dry to damp, loose, fine grained sand.
			NP	98	31	brown, damp, medium dense, some cementation, fine to medium grained sand.
			NP	96	25	
		26	2	100	29	medium grained sand.
			NP	98	20	medium to fine grained sand, no cementation.
			NP	98	17	
			NP	96	15	
			NP	99	38	some cementation.
			NP	98	28	
		26	2	100	33	dark brown.
22.0'	SC-SM	25	4	97	24	SILTY SAND CLAYEY SAND; brown, damp, no cementation.
24.0'	SM	29	4	99	30	SILTY SAND; brown, damp, slight cohesion, fine to medium grained sand.
25.0'						

TH85-12

DEPTH	LOG	LL	PI	-4	-200	DESCRIPTION
2.0'	SC-SM	26	6	94	20	SILTY SAND/CLAY SAND; light brown, dry, dense, slight cohesion.
4.0'	SM		NP	87	19	SILTY SAND; brown, moist, dense, no cohesion, cemented, few fine gravel.
8.0'	SC-SM	20	5	90	22	SILTY SAND CLAYEY SAND; brown, damp, very dense, few grained gravel.
			NP	97	18	SILTY SAND; brown, damp, dense, medium to fine grained sand.
	SM	32	8	89	27	few fine grained gravel to 3/4".
13.5'	SW-SM					SAND/SILTY SAND; brown, damp, very dense, few gravel to 1", graduation may be influenced by drifting bucket.
14.5'			NP	86	10	
15.5'	SM		NP	90	26	SILTY SAND; brown, damp, dense, slight cohesion, thin lens of weathered granite at 14.5', very dense at 15.5'.

TH85-13

DEPTH	LOG	LL	PI	-4	-200	DESCRIPTION
3.0'	SP-SM		NP	57	8	GRAVELLY SAND/SILTY GRAVELLY SAND; greyish brown, dry, loose, gravel to 2".
5.0'	SW-SM		NP	89	11	SAND/SILTY SAND; brown, damp.
6.0'	SM		NP	91	15	SILTY SAND; brown, damp, loose, slight cohesion.
9.0'	SW-SM		NP	100	12	SAND/SILTY SAND; brown, damp.
10.0'	SM		NP	86	21	SILTY SAND; brownish grey, damp, very dense, cemented, some cohesion from 9.5' to 10'.
10.5'	SC	28	9	76	25	
13.0'	SP-SM		NP	78	12	GRAVELLY CLAYEY SAND; grey and brown, damp, dense, some decomposed granite.
	SM		NP	91	18	GRAVELLY SAND/SILTY GRAVELLY SAND; brown with light grey, no cohesion, gravel to 2", decomposed granite.
			NP	93	17	
16.0'			NP	79	13	SILTY SAND; brown, very dense decomposed granite.
						brown and grey, damp, dense.
						SILTY GRAVELLY SAND; brown, damp, medium dense, very dense rocky material at 16'.

SCALE: IN. = 5 FT.

SCALE 5 0 5 10 15 FEET

NOTES:

1. SEE PLATE 60 FOR LOCATION OF TEST HOLES.
2.
3. TEST HOLES WERE DRILLED USING A 24" BUCKET AUGER IN JUNE 1985.

SYMBOL	DESCRIPTION	DATE

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

IMPERVIOUS BORROW AREA
SOIL LOGS
TH85-8 THRU TH85-14

SUBMITTED BY: DATE APPROVED:

CHIEF, BRANCH

SPEC. NO. DACW09- -B-

DISTRICT FILE NO.

PLA

SAFETY PAYS

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

TT85-1

DEPTH	LOG	LL	PI	-4	-200	DESCRIPTION
		14	1	97	27	SILTY SAND, brown, moist, medium dense, fine grained sand.
	SM	20	1	98	17	cobbles to 12"
7.5'		20	2	96	22	
	SW-SM		NP	84	5	GRAVELLY SAND/SILTY GRAVELLY SAND, brown, occasional subrounded cobbles to 6".
14.0'						

TT85-2

DEPTH	LOG	LL	PI	-4	-200	DESCRIPTION
		18	2	98	28	SILTY SAND, brown, dry, medium dense, fine grained sand
	SM	19	1	97	27	
		20	0	95	19	
10.0'		20	0	97	21	moist

DEPTH: 4.0', 10.0', 15.0'

TT85-5

DEPTH	LOG	LL	PI	-4	-200	DESCRIPTION
	SC-SM	25	5	97	43	SILTY SAND/CLAYEY SAND, reddish-brown, dry, dense.
4.0'	SC	20	8	97	40	CLAYEY SAND, reddish-brown, moist, dense.
7.0'	SC-SM	28	6	95	24	SILTY SAND/CLAYEY SAND, brown, moist, dense.
11.0'	SM	36	5	94	19	SILTY SAND, brown, moist, medium dense.
13.0'	SC-SM	26	6	97	42	SILTY SAND/CLAYEY SAND, brown, moist, dense.
15.5'						

TT85-6

DEPTH	LOG	LL	PI	-4	-200	DESCRIPTION
	SC-SM	21	5	98	29	SILTY SAND/CLAYEY SAND, brown, dry, loose, fine grained sand
4.0'		23	2	90	22	SILTY SAND, brown, dry, loose, few subrounded gravel and cobbles
	SM	22	3	96	32	dark brown, moist, dense
		21	2	99	20	occasional subandgular fine grained gravel
15.0'						

DEPTH: 5.0', 16.0'

TT85-9

DEPTH	LOG	LL	PI	-4	-200	DESCRIPTION
	SC	29	8	99	31	CLAYEY SAND, light brown, dry, loose
5.0'						
7.0'	SW-SM		NP	88	8	SAND/SILTY SAND, brown, dry, loose, few gravel
9.0'	SP-SM		NP	77	7	GRAVELLY SAND/SILTY GRAVELLY SAND, occasional cobbles to 12".

TT85-10

DEPTH	LOG	LL	PI	-4	-200	DESCRIPTION
	SM	37	8	91	27	SILTY SAND, reddish-brown, dry, occasional subrounded cobbles
4.0'						
	CL	39	17	97	55	SANDY CLAY, grey, decomposed granite boulders
		39	15	96	65	dark grey, moist
10.0'						

DEPTH: 2.5', 14.0'

TT85-13

DEPTH	LOG	LL	PI	-4	-200	DESCRIPTION
	SC	31	13	98	34	CLAYEY SAND, reddish-brown, dry, dense, few subangular to subrounded gravel
5.0'						
6.5'	SC-SM	26	5	95	17	SILTY SAND/CLAYEY SAND, reddish-brown, dry, dense, few subangular to subrounded gravel.
	SW-SM	26	1	89	10	SAND/SILTY SAND, reddish-brown, subangular to subrounded cobbles to 4".
12.0'			NP	86	8	
	SM	22	1	91	15	SILTY SAND, reddish-brown, one subrounded 10" cobble.
16.0'		21	3	88	17	brown.

TT85-14

DEPTH	LOG	LL	PI	-4	-200	DESCRIPTION
	SC	30	12	58	40	CLAYEY SAND, reddish brown, dry, dense
4.0'						
	SM	23	1	97	13	SILTY SAND, brown, dry, dense, few gravel (decomposed granite)
8.5'		24	3	99	20	
9.0'	SW-SM	24	1	95	11	SAND/SILTY SAND, brown, dry, dense.

PAYS

VALUE ENGINEERING PAYS

TT85-

PI	-4	-200	DESCRIPTION
2	98	47	SIL...
3	98	49	...SCRIPTION ...wn, dry, medium dense, ...nd.
5	98	45	SIL... moi...
5	97	40	bro...
2	98	28	SIL...
0	97	24	occ...

TT85-3

DEPTH	LOG	LL	PI	-4	-200	DESCRIPTION
		19	2	98	47	SILTY SAND; brown, dry, dense.
4.0'	SM	19	3	98	49	reddish-brown, very dense.
	SC-SM	23	5	98	45	SILTY SAND/CLAYEY SAND, dark brown, moist, very dense.
10.0'		25	5	97	40	brown, dense.
	SM	21	2	98	28	SILTY SAND; brown, moist, loose.
15.0'		20	0	97	24	occasional cobbles to 6".

TT85-4

DEPTH	LOG	LL	PI	-4	-200	DESCRIPTION
1.0'	SM	18	2	98	47	SILTY SAND, brown, dry, dense.
	SC-SM	25	6	97	49	SILTY SAND/CLAYEY SAND, reddish-brown, dry, very dense.
10.5'		23	4	99	38	moist, dense.
11.5'	SM	22	1	90	22	SILTY SAND, brown, moist, dense, occasional cobbles.

TT85-

PI	-4	-200	DESCRIPTION
5	99	41	SIL...
5	100	47	...EY SAND, brown, dry, loose, ...and.
2	99	36	SIL... ...rown, dry, loose, few subrounded ...bles. ...ist, dense.
2	96	22	
3	97	24	bro... subangular fine grained gravel.
3	95	29	occ...

TT85-7

DEPTH	LOG	LL	PI	-4	-200	DESCRIPTION
	SC-SM	20	5	99	41	SILTY SAND/CLAYEY SAND; brown, dry, loose, fine grained sand.
5.0'		22	5	100	47	reddish-brown, moist, medium dense.
	SM	22	2	99	36	SILTY SAND; reddish- brown, damp, medium dense.
		22	2	98	22	
		24	3	97	24	brown, few subangular gravel, occasional cobbles to 8".
16.0'		24	3	95	29	occasional cobbles to 10".

TT85-8

DEPTH	LOG	LL	PI	-4	-200	DESCRIPTION
1.0'	SM	19	3	99	45	SILTY SAND, brown, dry, medium dense.
4.0'	CL	24	8	98	50	SANDY CLAY, reddish-brown, dry, dense.
7.0'	SC-SM	25	7	71	20	SILTY SAND/CLAYEY SAND, reddish-brown, moist, dense.
	SM	24	2	78	15	SILTY SAND; brown, moist, loose, occasional subangular cobbles to 7".
15.0'		21	2	95	20	

TT85-

PI	-4	-200	DESCRIPTION
5	97	45	SIL... dry,
11	97	43	...DESCRIPTION CLAY... ...reddish-brown, dry, occasional ...cobbles.
10	96	33	...grey, decomposed granite ...ist
8	97	26	occ... gran...

TT85-11

DEPTH	LOG	LL	PI	-4	-200	DESCRIPTION
2.5'	SC-SM	22	5	97	45	SILTY SAND/CLAYEY SAND; reddish-brown, dry, medium dense.
	SC	30	11	97	43	CLAYEY SAND, reddish-brown, dry, dense.
		28	10	96	33	
14.0'		27	8	97	26	occasional gravel, refusal at 14', granite.

TT85-12

DEPTH	LOG	LL	PI	-4	-200	DESCRIPTION
	SW-SM		NP	89	5	SAND/SILTY SAND, brown, dry.
6.0'			NP	64	7	GRAVELLY SAND/SILTY GRAVELLY SAND, brown, dry, occasional subrounded cobbles to 6".

DESCRIPTION

...0'. reddish-brown, dry, dense

...brown, dry, dense, few gravel
...d granite).

SAND, brown, dry, dense.

S:

SEE PLATE 60

AND CLASSIFICAT...

TEST TRENCHES W...
JUNE 1985.

NOTES:

1. SEE PLATE 60 FOR LOCATION OF TEST TRENCHES.
2. ...AND CLASSIFICATION SYSTEM.
3. TEST TRENCHES WERE EXCAVATED BY BACKHOE IN JUNE 1985.

SCALE IN = 5 FT

SCALE 5 0 5 10 15 FEET

SYMBOL	DESCRIPTIONS	DATE	APP'R
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

IMPERVIOUS BORROW AREA
SOIL LOGS
TT85-1 THRU TT85-14

SUBMITTED BY: DATE APPROVED:

CHIEF, ENGR. DIV.

SPEC. NO. DACW09-___-B-____

DISTRICT FILE NO.

PLAT

SAFETY PAYS

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

CBH 86 - 1

DEPTH	LOG	MC NO	LL SAMPLE	PI TAKEN	-4	-200	DESCRIPTION
							SILTY SAND, brown, dry, top soil
	SM			NP	83	21	SILTY SAND WITH GRAVEL, red-brown, dry, gravel to 3", few cobbles to 4".
5.0'			25	3	85	16	
	SP-SM		23	2	79	12	POORLY GRADED SAND WITH SILT AND GRAVEL, yellow-brown, bucket breaking up cobbles, drilling stopped due to large rocks
8.0'							

CBH 86 - 2

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
7.0'	SC		29	8	96	31	CLAYEY SAND, red-brown, damp, 10 % cobbles to 5", many boulders on surface to 4'.
	SC-SM		26	7	96	23	SILTY CLAYEY SAND, red-brown, slightly damp, gravel to 2", 3" cobble at 4'.
4.0'							
	SM	4	23	2	86	15	SILTY SAND, light brown, slightly damp, gravel to 2".
10.0'			21	1	88	15	
	SC-SM	5	20	4	89	15	SILTY, CLAYEY SAND, light brown, slightly damp, medium dense, gravel to 3".
13.0'							
	SM			NP	89	16	SILTY SAND, light brown, slightly damp, medium dense, gravel to 3"
				NP	8	16	
18.0'							
			23	4	86	18	SILTY, CLAYEY SAND, light brown, slightly damp, dense, few cobbles to 6".
			26	5	79	24	SILTY, CLAYEY SAND WITH GRAVEL, light red-brown, slightly damp, lightly cemented gravel to 3".
	SC-SM		25	7	87	14	SILTY, CLAYEY SAND, light red-brown, slightly damp, few cobbles to 4".
			21	4	92	14	
30.0'					91	18	yellow-brown, damp, slight cohesion, stopped drilling due to rocks at 30'.

CBH 86 - 3

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
3.0'	CL		43	25	100	52	SANDY CLAY, red-brown, slightly damp, lightly cemented, 10 % cobbles and few boulders on surface, to 4'.
5.0'	SM		28	1	85	21	SILTY SAND WITH GRAVEL, light brown, dry, stopped drilling due to rocks moved hole 3' and still couldn't penetrate rock layer.

CBH 86 - 4

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
			27	4	91	28	SILTY SAND, red-brown, damp, gravel to 3", one 9" cobble, boulders on surface to 5'
			30	6	66	18	SILTY SAND WITH GRAVEL, red-brown, damp, some cemented clods, gravel to 2".
			24	3	72	13	light brown, 10 % cobbles to 5".
	SM			NP	76	15	
				NP	71	16	dry, gravel to 2", few cobbles to 4".
		3	22	1	75	15	gravel to 3", few cobbles to 6".
				NP	88	15	SILTY SAND, light brown, slightly damp, gravel to 3", few cobbles to 4".
18.0'				NP	83	13	SILTY SAND WITH GRAVEL, light brown, slightly damp, gravel to 3", stopped drilling due to large rock.

CBH 86 - 5

	LOG	MC	LL	PI	-4	-200	DESCRIPTION
			41	11	98	43	SILTY SAND, red-brown, damp, slightly cemented
	SM						
5.0'			28	4	99	24	less cohesive.
8.0'	SC		24	9	95	14	CLAYEY SAND, light brown, dry, slightly cemented, gravel to 2".
10.0'	SM		23	2	96	15	SILTY SAND, light brown, damp, slightly cemented.
	SC-SM		25	5	100	26	SILTY, CLAYEY SAND, light brown, damp, slightly cemented, occasional gravel to 5".
13.0'							
16.0'	SW-SM	3		NP	94	11	WELL GRADED SAND WITH SILT, light brown, slightly damp, few cobbles to 5".
				NP	85	14	SILTY SAND WITH GRAVEL, light brown, damp, moderately cemented clods, few cobbles to 4"
	SM			NP	96	13	SILTY SAND, light brown, damp, moderately cemented clods, few cobbles to 4"
22.0'							
23.0'	SC-SM	5	26	5	98	27	SILTY, CLAYEY SAND, brown, damp, slightly cemented
25.0	SW-SM			NP	79	8	WELL GRADED SAND WITH SILT AND GRAVEL, brown, damp, gravel to 2".
	SM		23	2	80	14	SILTY SAND WITH GRAVEL, brown, damp, gravel to 2".
28.0'							
	SW-SM			NP	89	12	WELL GRADED SAND WITH SILT, red-brown,
31.0		5		NP	89	9	damp, encountered rocks at bottom of hole.

CBH 86-6

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
	SC		31	11	89	20	CLAYEY SAND, red-brown, dry, slightly cemented, gravel to 1", few boulders on surface to 2'.
3.0'							
5.0'	SP-SC		25	4	70	12	POORLY GRADED SAND WITH SILTY CLAY AND GRAVEL, light brown, dry, 10 % cobbles to 8".
7.0'	SW-SM		28	1	85	11	WELL GRADED SAND WITH SILT AND GRAVEL, light brown, dry, 10 % cobbles to 5" drilling stopped due to rock at 7.5'.

DEPTH	LOG
	SC
6.0'	
	SP-SM
13.0'	
	SM
16.0'	
17.0'	SW-SC
	SM
20.0'	
	SP-SC
23.0'	
	SM
28.0'	
30.0'	SP-SM
	SM
39.5'	
	SC
45.0'	
48.0'	SW-SM
	SM
56.0	
58.0	SC
	SM
61.0'	

YS

VALUE ENGINEERING PAYS

CBH

-4 -200

			DESCRIPTION
	99	25	Cl... clo...
	90	23	Y... SAND; red-brown, damp, gravel to 3". ...bble, boulders on surface to 5'
	87	12	POO... SAND WITH GRAVEL, red-brown, damp. ...mented clods, gravel to 2".
P	71	9	POO... red... brown, 10 % cobbles to 5".
P	86	10	POO... to...
	86	14	POO... gra... Sil...
	86	11	...avel to 2", few cobbles to 4".
	87	13	WEL... bro... to 3", few cobbles to 6".
	78	12	Sil... SAND; light brown, slightly damp, ...to 3", few cobbles to 4".
P	91	18	POO... SAND WITH GRAVEL; light brown, ...ly damp, gravel to 3", stopped ...ng due to large rock.
	98	27	Sil...
	76	10	man...
			POO... bro...
IP	86	14	Sil... cla...
4P	92	14	few... roc...
NP	91	18	det...
			CLA...
14	96	24	...

6 - 5

			DESCRIPTION
NP	91	11	WEL... mer...
3	98	22	SIL... SAND; red-brown, damp, slightly ...ed ... to ...cohesive.
NP	91	17	bro... SAND; light brown, dry, slightly ...ed, gravel to 2".
'LE	TAKEN		mon...
11	98	36	SAND; light brown, damp, slightly ...ed.
			CLAYEY SAND, light brown, damp, ...tly cemented, occasional gravel
1	84	20	Sil... gra...
1	76	20	GRADED SAND WITH SILT, light brown, ...tly damp, few cobbles to 5".
			SAND WITH GRAVEL, light brown, ...oderately cemented clods, few ...es to 4".
			SAND, light brown, damp, moderately ...nted clods, few cobbles to 4".
			, CLAYEY SAND; brown, damp, slightly ...nted.
			GRADED SAND WITH SILT AND GRAVEL, ...n, damp, gravel to 2".
			SAND WITH GRAVEL, brown, damp, gravel ...
			GRADED SAND WITH SILT, red-brown, ...encountered rocks at bottom of hole.

86 - 6

DESCRIPTION
SAND; red-brown, dry, slightly co... gravel to 1", few boulders on sur... e 3'.
GRADED SAND WITH SILTY CLAY AND ... light brown, dry, 10 % cobbles to 6".
RADED SAND WITH SILT AND GRAVEL, ...brown, dry, 10 % cobbles to 5" ...ng stopped due to rock at 7.5'.

CBH 86 - 7

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
		19	32	12	99	25	CLAYEY SAND, red-brown, damp, many 2"-5" clods, cohesive.
	SC						
			29	11	90		yellow-brown, cohesive, gravel to 2"
8.0'							
			28	4	87	12	POORLY GRADED SAND WITH SILT, yellow-brown, damp, cohesive, few gravel to 2"
	SP-SM			NP	71	9	POORLY GRADED SAND WITH SILT AND GRAVEL, red-brown, cohesive, gravel to 2", few cobbles to 4"
				NP	86	10	POORLY GRADED SAND WITH SILT, red-yellow-brown, gravel to 3".
13.0'							
	SM		25	3	86	14	SILTY SAND, red-yellow-brown, gravel to 3"
16.0'							
17.0'	SW-SC		22	5	86	11	WELL GRADED SAND WITH SILTY CLAY, red-yellow-brown, gravel to 3", few cobbles to 4"
	SM		26	4	87	13	SILTY SAND, red-brown, gravel to 3"
20.0'							
	SP-SC	7	25	5	78	12	POORLY GRADED SAND WITH SILTY CLAY AND GRAVEL, red-brown, damp, cemented clods.
23.0'							
				NP	91	18	SILTY SAND, red-brown, damp, cemented clods.
	SM						
		8	27	2	98	27	many cemented clods.
28.0'							
30.0'	SP-SM		22	3	78	10	POORLY GRADED SAND WITH SILT AND GRAVEL, red-brown, slightly damp, cemented clods.
							SILTY SAND, red-brown, slightly damp, cemented clods.
				NP	86	14	few cemented clods with decomposed angular rock, few cobbles to 6".
	SM			NP	92	14	dense, cemented clods.
				NP	91	18	
39.5'							
							CLAYEY SAND, light red-brown, damp.
	SC		24	14	96	24	
45.0'							
	SW-SM			NP	91	11	WELL GRADED SAND WITH SILT, red-brown, cemented clods.
48.0'							
		10	25	3	98	22	SILTY SAND, red-brown, damp, alternating layers and lenses of sand and clayey silt, gravel to 1"
	SM			NP	91	17	brown, damp, many clods, cohesive.
56.0		NO	SAMPLE		TAKEN		many cemented clods.
58.0	SC	11	24	11	98	36	CLAYEY SAND, red-brown, damp.
	SM		23	1	84	20	SILTY SAND WITH GRAVEL, red-brown, few c...ds gravel to 1".
61.0'			27	1	76	20	fragmented gravel and cobbles to 5", probably broken from larger rocks by bucket drilling stopped due to rocks

CBH 86 - 8

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
3.0'	SC		36	16	98	25	CLAYEY SAND, red-brown, damp, small clods
			29	7	86	21	SILTY, CLAYEY SAND, red-brown, damp, small clods, gravel to 2".
7.0'	SC-SM		24	4	91	18	light brown
				NP	80	12	POORLY GRADED SAND WITH SILT AND GRAVEL, light brown, dry, slight cohesion, gravel to 3", few cobbles to 4"
	SP-SM						few cobbles to 5".
				NP	79	9	10 % cobbles to 5".
16.0'							
		24	2	91	14		SILTY SAND; light brown, dry, slight cohesion, few cobbles to 5".
		5					slightly damp.
	SM			NP	90	15	few small clods, gravel to 2", few cobbles to 5".
				NP	96	16	red-brown 15 % small clods, no cobbles encountered.
25.0'							
26.0'	SW-SM			NP	88	10	WELL GRADED SAND WITH SILT, red-brown, few clods.
28.0'	SC		23	18	94	14	CLAYEY SAND, red-brown, few clods.
	SM		26	11	97	21	SILTY SAND, red-brown, damp, 30 % small clods.
		7	24	3	94	16	
34.0							
	SC-SM		24	4	96	19	SILTY, CLAYEY SAND, red-brown, damp, 20 % clods, gravel to 1".
37.0'							
			29	5	98	35	SILTY SAND, light brown, damp, gravel to 1/2".
	SM		27	2	98	20	red-brown, dense, 15 % clods, gravel to 1".
43.0		4	37	11	98	39	gravel to 2"
			26	7	96	19	SILTY, CLAYEY SAND, dark red-brown, damp, clods, gravel to 2", cohesive.
	SC-SM		26	6	97	21	dark brown, clods.
48.0							
			34	14	99	27	CLAYEY SAND, dark brown, damp, dense, many clods.
	SC		30	9	96	17	red-brown, many clods
53.0							
			24	3	88	12	POORLY GRADED SAND WITH SILT; red-brown, damp, many clods, gravel to 2"
			25	3	86	12	brown, gravel to 3"
	SP-SM			NP	79	8	POORLY GRADED SAND WITH SILT AND GRAVEL, red-brown, damp, gravel to 3".
				NP	69	9	brown, few cobbles to 4".
65.0							drilling stopped due to large rock

SCALE (IN FEET)

SCALE 5 0 5 10 15 FEET

SYMBOL	DESCRIPTIONS	DATE	APPD
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY

DRAWN BY F. CASTILLAJA

CHECKED BY

SANTA ANA RIVER CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

IMPERVIOUS BORROW AREA
SOIL LOGS
CBH 86-1 THRU CBH 86-8

SUBMITTED BY

DATE APPROVED

SPEC. NO. DACW09-___-___

DISTRICT FILE NO

PLATE

SAFETY PAYS

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

CBH 86-9

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
	SM			NP	100	33	SILTY SAND, orange-red-brown, dry, 30% small clods, gravel to 1/2", few boulders on surface to 3'.
3.0'							
4.0'	SC-SM		22	5	79	15	SILTY, CLAYEY SAND WITH GRAVEL, light brown, dry, angular gravel to 2" possibly fractured from rock by bucket, could not advance past 4' depth due to rock, moved drilling 4' to new location, begin logging again at 4'.
6.0'	SW-SM			NP	79	9	WELL GRADED SAND WITH SILT AND GRAVEL; light brown gravel to 3".
							drilling stopped due to large rock.

CBH 86-11

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
2.0'	SC-SM		28	6	100	37	SILTY, CLAYEY SAND, red-brown, dense, cemented.
5.0'	SM		30	4	74	13	SILTY SAND WITH GRAVEL, light brown, damp, gravel to 3".
				NP	72	8	POORLY GRADED SAND WITH SILT AND GRAVEL, light brown, damp, gravel to 3",
10.5'	SP-SM			NP	74	9	few cobbles to 5".
13.0'	SW-SM			NP	78	11	WELL GRADED SAND WITH SILT AND GRAVEL, light brown, damp, angular gravel and cobbles to 5".
				NP	68	11	POORLY GRADED SAND WITH SILT AND GRAVEL,
	SP-SM			NP	77	10	light brown, damp, angular gravel and few angular cobbles to 6"
18.5'				NP	72	10	with streaks of greenish sand, few cobbles to 9"
							drilling stopped due to large rock.

CBH 86-10

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
				NP	86	7	POORLY GRADED SAND WITH SILT, red-brown, damp.
	SP-SM			NP	84	7	POORLY GRADED SAND WITH SILT AND GRAVEL,
4.0'							light red-brown, dry, gravel to 1".
	SW-SM			NP	87	7	WELL GRADED GRAVEL WITH SILT, light red-brown, dry, gravel to 2".
6.5'							
	SP-SM			NP	70	7	POORLY GRADED SAND WITH SILT AND GRAVEL, light brown, gravel to 3".
9.0'							
				NP	81	6	WELL GRADED SAND WITH SILT AND GRAVEL, light brown with streaks of reddish fines, gravel to 3".
	SW-SM						
				NP	82	6	few cobbles to 4".
15.0'							
				NP	94	13	SILTY SAND, red-brown, few small clods,
	SM						
			33	9	95	14	damp, few cobbles to 4".
20.0'							
21.0'	SW-SM			NP	92	7	WELL GRADED SAND WITH SILT, light brown,
							damp, gravel to 1/2".
24.0'	SP-SM			NP	87	10	POORLY GRADED SAND WITH SILT, light brown, gravel to 2".
							WELL GRADED SAND WITH SILT, light brown,
	SW-SM			NP	87	6	gravel to 2".
				NP	90	10	small clods, gravel to 3/4".
29.0'							
	SP-SM			NP	85	9	POORLY GRADED SAND WITH SILT AND GRAVEL, light brown, damp, gravel to 2".
32.0'							
34.0'	SM		27	4	97	14	SILTY SAND, red-brown, damp, few large clods.
36.0'	SP-SM			NP	89	11	POORLY GRADED SAND WITH SILT, light brown, gravel to 2".
			25	2	94	13	SILTY SAND, light brown, few small clods,
	SM	11	27	5	97	16	light red-brown, many small clods,
			28	4	97	13	red-brown.
42.0'							
43.0'	SP-SM			NP	88	9	POORLY GRADED SAND WITH SILT, light red-brown, damp, few cobbles to 5".
	SM			NP	83	15	SILTY SAND WITH GRAVEL, light red-brown,
46.0'							damp, gravel to 3", few cobbles to 6".
48.0'	SC		34	12	99	47	CLAYEY SAND, red-brown, many small clods.
			30	4	62	18	SILTY SAND WITH GRAVEL, red-brown, gravel to 3".
	SM		38	11	100	42	SILTY SAND, red-brown with dark brown fines,
52.0'							clods to 10" have shiny sides, cohesive.
	SC		44	21	100	44	CLAYEY SAND, red-brown with dark brown fines and streaks of reddish sand, cohesive,
55.0'							clods to 10" have shiny sides.
	SC-SM	8	25	4	78	15	SILTY, CLAYEY SAND WITH GRAVEL, red-brown,
58.0'							gravel to 2", few cobbles to 6",
							drilling stopped due to rock

CBH 86-12

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
1.0'	SC		33	15	98	58	CLAYEY SAND, red-brown, damp.
2.5'	CH		58	31	100	58	SANDY FAT CLAY, red-brown, damp, very hard.
4.5'	SC		33	14	92	34	CLAYEY SAND, light brown, gravel to 1-1/2", few cobbles to 5"
7.0'	SM			NP	92	13	SILTY SAND, yellow-brown, gravel to 3".
9.5'	SW-SM			NP	78	9	WELL GRADED SAND WITH GRAVEL, light brown, few cobbles to 4".
				NP	72	12	POORLY GRADED SAND WITH SILT AND GRAVEL, brown, damp, gravel to 1-1/2", 5% cobbles to 5".
	SP-SM			NP	65	8	gravel to 3" fractured from cobbles to 6".
		3		NP	70	9	
				NP	80	8	light brown, damp
20.0'							
21.0'	SM		22	3	61	24	SILTY SAND WITH GRAVEL, light brown, damp, gravel to 3", 1-10" cobble
							drilling stopped due to rock.

CPH 86-13

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
3.0'	SC		38	16	97	32	CLAYEY SAND, red-brown, damp, cohesive.
6.0'	SP-SM			NP	98	7	POORLY GRADED SAND WITH SILT, red-brown, damp, gravel to 3".
8.0'	SW-SM			NP	99	6	WELL GRADED SAND WITH SILT, brown, damp, slightly cohesive.
9.0'	SP-SM			NP	82	7	POORLY GRADED SAND WITH SILT AND GRAVEL,
				NP	97	13	brown, damp, slightly cohesive, gravel to 3".
							SILTY SAND, brown, damp, slightly cohesive
			25	2	95	18	small clods
	SM		28	4	93	17	
			34	10	94	21	clods to 6"
			31	4	90	21	
24.0'			32	8	93	20	SILTY SAND, brown, damp, cohesive, gravel to 3", few cobbles to 6"
							drilling stopped due to large rock.

'AYS

LUE ENGINEERING PAYS

CBH

PI	-4	-200	
3	93	23	
NP	93	17	
6	100	38	
NP	86	14	
4	88	23	
3	88	27	
NP	92	15	
1	82	14	
NP	82	8	
NP	81	11	

ON
red-brown, dense,
VEL, light brown,
WITH SILT AND GRAVEL, gravel to 3".
ITH SILT AND GRAVEL, angular gravel and
WITH SILT AND GRAVEL, angular gravel and to 6"
reenish sand, low
due to large rock.

CBH

L	PI	-4	-200	
8	13	95	34	
10	9	93	25	
	NP	89	8	
	NP	84	8	
	NP	89	9	
	NP	68	8	
	NP	80	7	
	NP	91	7	
	NP	85	14	
	NP	86	7	
	NP	87	9	
8	10	89	23	
7	4	98	16	
4	7	96	13	
	NP	94	18	
	NP	78	10	
7	4	97	13	
8	7	95	16	

PTION
brown, damp.
red-brown, damp, very
ght brown, gravel to
bles to 5"
low-brown, gravel to
D WITH GRAVEL, light
bles to 4".
AND WITH SILT AND GRAVEL, avel to 1-1/2", 5% cobbles
actured from cobbles to 5".
mp
TH GRAVEL, light brown, damp, 1-10" cobble
ed due to rock.

CRIPTION
red-brown, damp, cohesive.
SAND WITH SILT, red-brown, to 3".
AND WITH SILT, brown, damp, sive.
SAND WITH SILT AND GRAVEL, slightly cohesive, gravel
brown, damp, slightly cohesive
brown, damp, cohesive, gravel bbles to 6"
oped due to large rock.

CBH 86-15

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
	SM		25	3	93	23	SILTY SAND, light brown, damp, slight cohesion.
				NP	93	17	medium dense, gravel to 1".
			31	6	100	38	dense.
10.0'				NP	86	14	
11.0'	SC-SM		23	4	88	23	SILTY, CLAYEY SAND, light brown, damp, gravel to 1-1/2".
			23	3	88	27	SILTY SAND, brown, damp.
	SM			NP	92	15	
18.0'			22	-	82	14	SILTY SAND WITH GRAVEL, light brown, damp, slight cohesion, gravel to 1-1/2"
	SW-SM			NP	82	8	WELL GRADED SAND WITH SILT AND GRAVEL, light brown, damp, slight cohesion, gravel to 1-1/2".
				NP	81	11	gravel to 3".
23.0'							drilling stopped due to rock.

CBH 86-14

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
	SM		38	13	95	34	SILTY SAND, red-brown, damp, cohesive, hard clods.
3.0'							
	SC		30	9	93	25	CLAYEY SAND, brown, damp, cohesive.
6.0'							
				NP	89	8	POORLY GRADED SAND WITH SILT, brown, damp, slightly cohesive.
				NP	84	8	POORLY GRADED SAND WITH SILT AND GRAVEL, brown, damp, slightly cohesive, gravel to 1-1/2".
	SP-SM			NP	88	9	POORLY GRADED SAND WITH SILT, brown, damp, slightly cohesive
				NP	88	8	POORLY GRADED SAND WITH SILT AND GRAVEL, brown, damp, slightly cohesive, gravel to 2".
				NP	80	7	
18.0'							
20.0'	SW-SM			NP	91	7	WELL GRADED SAND WITH SILT, brown, damp, slightly cohesive
	SM			NP	85	14	SILTY SAND WITH GRAVEL, brown, damp.
23.0							
	SP-SM			NP	86	7	POORLY GRADED SAND WITH SILT, brown, damp, slightly cohesive.
26.5'				NP	87	9	
	SC	6	28	10	89	23	CLAYEY SAND, red-brown, damp, cohesive, large clods.
29.5'							
			27	4	98	16	SILTY SAND, red-brown, damp, slightly cohesive, large clods
	SM		34	7	96	13	no clods
				NP	94	18	
36.0'							
	SP-SM			NP	78	10	POORLY GRADED SAND WITH SILT AND GRAVEL, red-brown, damp, slight cohesion, gravel to 2".
38.0'							
41.0'	SM		27	4	97	13	SILTY SAND, red-brown, damp, slight cohesion, small clods.
43.0'	SC-SM		28	7	95	16	SILTY, CLAYEY SAND, red-brown, damp, slight cohesion, small clods.
							drilling stopped due to rock.

CBH 86-16

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
	SC-SM		24	5	93	36	SILTY, CLAYEY SAND, light brown, damp, medium dense, cohesive clods.
3.0'							
	SW-SM		22	2	95	12	WELL GRADED SAND WITH SILT, light brown, dry, medium dense,
6.0'							
	SM		22	1	86	17	SILTY SAND, light brown, dry, cohesive clods, gravel to 1-1/2".
9.0'							
	SW-SM		24	2	89	12	WELL GRADED SAND WITH SILT, light brown, dry.
11.0'							
				NP	92	22	SILTY SAND, light brown, dry, cohesive clods,
	SM			NP	93	15	brown
		S	22	1	92	23	slightly damp, cohesive.
19.0'							
	SW-SM			NP	94	11	WELL GRADED SAND WITH SILT, light brown, slightly damp, gravel to 3".
22.0							
			23	2	96	30	SILTY SAND, light brown, slightly damp.
	SM		24	3	85	18	SILTY SAND WITH GRAVEL, light brown, damp.
			22	2	88	22	SILTY SAND, brown with traces of red, moist, medium dense, cohesive.
30.0'							
33.0'	SW-SM			NP	79	11	WELL GRADED SAND WITH SILT AND GRAVEL, brown, damp, gravel to 2", few decomposing cobbles.
35.0'	SC-SM		26	5	88	28	SILTY, CLAYEY SAND, brown, damp, cohesive, few clods, gravel to 2".
			26	3	88	22	SILTY SAND, brown, damp, slightly cohesive.
	SM		24	2	88	37	
40.0'							
	SC-SM	7	26	5	85	28	SILTY, CLAYEY SAND, brown, damp, slightly cohesive.
43.0'							
			23	1	88	29	SILTY SAND, brown, damp, slightly cohesive.
	SM		23	2	92	22	
			23	1	91	32	
51.0'							
			24	7	97	27	SILTY, CALYEY SAND, dark brown, damp, slightly cohesive.
	SC-SM		24	4	90	21	
56.0'			22	4	90	14	
	SM		29	6	96	41	SILTY SAND, dark brown, damp, cohesive, few subround gravel
60.0'			24	1	97	33	drilling stopped due to rock.

SCALE 1 IN = 5 FT

SCALE 5 0 5 10 15 FEET

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY: I.F.B.
CHECKED BY:

SANTA ANA RIVER, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

IMPERVIOUS BORROW AREA
SOIL LOGS
CBH 86-9 THRU CBH 86-16

SUBMITTED BY: DATE APPROVED:
CHIEF, BRANCH

SPEC. NO. DACW09- - -
DISTRICT FILE NO.

SHEET

PLATE A-66

SAFETY PAYS

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

CBH 86-17

DEPTH	LOG	MC	LL	PI	-4	-200	
			26	2	93	21	SILTY SAND; brown, moist, no cohesion, subrounded gravel to 1".
			25	3	92	13	damp.
				NP	90	22	dry
				NP	92	16	rounded to subrounded gravel to 1".
				NP	86	13	gravel to 1-1/2", occasional cobble.
	SM			NP	74	13	SILTY SAND WITH GRAVEL, brown, dry, no cobbles encountered.
				NP	94	14	SILTY SAND, brown, damp, rounded to subrounded gravel to 3"
			24	2	90	15	
			24	1	86	15	
30.0'			23	3	88	13	gravel to 1".
				NP	86	10	WELL GRADED SAND WITH SILT, brown, damp.
	SW-SM			NP	90	11	pieces of decomposed rock to 4".
37.0'				NP	79	9	gravel to 3", no decomposed rock encountered.
	SM		27	1	94	16	SILTY SAND; brown, damp, slightly cohesive, gravel to 1".
41.0'			24	2	96	23	
44.0'	SC-SM		25	7	90	16	SILTY, CLAYEY SAND, brown, damp, slightly cohesive, gravel to 1-1/2".
	SW-SM			NP	71	11	WELL GRADED SAND WITH SILT AND GRAVEL, brown, damp, gravel to 3".
50.0'				NP	70	9	

CBH 86-18

DEPTH	LOG	MC	LL	PI	-4	-200	
3.0'	SC		36	15	99	34	CLAYEY SAND, red-brown, moist, medium dense to dense, subangular gravel to 1".
	SW-SM			NP	97	7	WELL GRADED SAND WITH SILT, red-brown, dry, slightly cemented clods.
9.0'				NP	98	12	brown, damp, no clods.
	SP-SM			NP	87	7	POORLY GRADED SAND WITH SILT; brown, slightly damp, subrounded gravel to 1".
15.0'				NP	93	7	damp, medium dense.
18.0'	SW-SM			NP	96	8	WELL GRADED SAND WITH SILT, brown, damp, medium dense, gravel to 1".
	SP-SM			NP	83	6	POORLY GRADED SAND WITH SILT AND GRAVEL, brown, damp, gravel to 3".
23.0'				NP	91	9	POORLY GRADED SAND WITH SILT; brown, damp.
26.0'	SP-SC	9	26	7	80	12	POORLY GRADED SAND WITH SILTY CLAY AND GRAVEL; red-brown, moist, slightly cohesive, subrounded gravel to 1-1/2".
	SC-SM		28	7	94	26	SILTY, CLAYEY SAND; red-brown, moist, cohesive, large clods.
32.0'			28	7	93	26	
	SP-SM			NP	86	9	POORLY GRADED SAND WITH SILT AND GRAVEL, red-brown, damp
				NP	78	7	brown, gravel to 3".
37.0'				NP	89	11	POORLY GRADED SAND WITH SILT; brown, damp, gravel to 3".
39.0'	SW-SM		27	4	92	12	WELL GRADED SAND WITH SILT, brown, damp, gravel to 3".
42.0'	SC-SM		28	6	95	14	SILTY, CLAYEY SAND; red-brown, damp.
45.0'	SW-SM			NP	83	12	WELL GRADED SAND WITH SILT AND GRAVEL; red-brown, damp, gravel to 1".
47.0'	SP-SM			NP	89	9	POORLY GRADED SAND WITH SILT AND GRAVEL; brown, damp, gravel to 1-1/2".
50.0'	SW-SM			NP	87	11	WELL GRADED SAND WITH SILT, brown, damp, gravel to 1".

CBH 86-19

DEPTH	LOG	MC	LL	PI	-4	-200	
			26	3	90	28	SILTY SAND, red-brown, wet, cohesive.
4.5'	SM		29	5	62	18	SILTY SAND WITH GRAVEL, red-brown, wet, lightly cemented, subrounded gravel to 1-1/2", larger rock, boulders and cobbles fractured by drill action to 3" to 6" size.
6.0'	SC-SM		27	6	81	16	SILTY, CLAYEY SAND WITH GRAVEL, brown, dry, slightly cemented.
8.0'	SW-SM			NP	70	8	WELL GRADED SAND WITH SILT AND GRAVEL, brown, dry, subrounded gravel to 1-1/2", 5 % cobbles to 6".
10.5'	SP-SM			NP	62	10	POORLY GRADED SAND WITH SILT AND GRAVEL, brown dry, subrounded gravel to 1-1/2", 10 % cobbles to 6"
12.0'	SM			NP	76	13	SILTY SAND WITH GRAVEL, brown, dry, subrounded gravel to 1-1/2"

DEPTH	LOG
3.0'	SM
7.5'	SC-SM
13.0'	SM
19.0'	SC-SM

CBH 86-20

DEPTH	LOG	MC	LL	PI	-4	-200	
2.0'	SC		30	10	90	21	CLAYEY SAND, dark brown, moist, cohesive few subangular to subrounded gravel to 1".
5.0'	SC-SM		27	7	100	19	SILTY, CLAYEY SAND, brown, dry, medium, dense, cemented clods, gravel to 1".
8.0'	SP-SM			NP	75	10	POORLY GRADED SAND WITH SILT AND GRAVEL, brown, dry, no clods, gravel to 3", 10 % cobbles to 6"
	SM			NP	79	14	SILTY SAND WITH GRAVEL, brown, dry, gravel to 1-1/2", few cobbles to 6".
12.0'			24	[illegible]	89	24	SILTY SAND, brown, dry, few cobbles to 6".

DEPTH	LOG
2.0'	SC
4.5'	SW-SI
8.5'	SM
11.0'	SW-S
13.0'	SP-S

VALUE ENGINEERING PAYS

CBH

	-4	-200	
	87	29	-brown, wet, cohesive.
	87	30	GRAVEL, red-brown, wet, d, subrounded gravel to ock, boulders and cobbles ill action to 3" to 6" size.
	76	25	ND WITH GRAVEL, brown, dry, ed.
	90	29	D WITH SILT AND GRAVEL, unded gravel to 1-1/2", 6".
	91	29	AND WITH SILT AND GRAVEL, brown gravel to 1-1/2", 10 %
	84	23	GRAVEL; brown, dry, sub- to 1-1/2".
	86	19	

CBH

PI	-4	-200	
8	92	14	ark, brown, moist, cohesive to subrounded gravel to".
NP	72	9	AND, brown, dry, medium, dense, gravel to 1".
1	81	13	SAND WITH SILT AND GRAVEL, lods, gravel to 3", 10 % cobbles
NP	72	11	TH GRAVEL, brown, dry, gravel to bles to 6".
NP	71	9	rown, dry, few cobbles to 6"

CBH 86-21

DEPTH	LOG	MC	LL	PI	-4	-200	
3.0'	SM		22	3	87	29	SILTY SAND; brown, slightly damp, non cohesive, easy drilling, gravel to 2".
	SC-SM		24	5	87	30	SILTY, CLAYEY SAND; red-brown, slightly damp, very dense
7.5'			22	5	76	25	SILTY SAND, red-brown, slightly damp, gravel to 1", few cobbles to 5".
	SM	6	23	1	90	29	
13.0'			21	3	91	29	streaks of dark brown, damp, flous-like in texture, gravel to 1-1/2".
	SC-SM		20	4	84	23	SILTY, CLAYEY SAND WITH GRAVEL; red-brown, damp, cobbles at 15', gravel to 3".
19.0'			22	5	86	19	SILTY, CLAYEY SAND; brown, damp, drilling fractures whitish-grey, rock into smaller sizes

CBH 86-22

DEPTH	LOG	MC	LL	PI	-4	-200	
2.0'	SC		26	8	92	14	CLAYEY SAND, brown, moist, non-cohesive at 1 foot depth hard drilling.
4.5'	SW-SM			NP	72	9	WELL GRADED SAND WITH SILT AND GRAVEL, light brown, moist, non cohesive subangular cobbles to 5", hard drilling.
8.5'	SM		23	1	81	13	SILTY SAND WITH GRAVEL, light brown, slightly moist, non cohesive, subangular cobbles to 8".
11.0'	SW-SM			NP	72	11	WELL GRADED SAND WITH SILT AND GRAVEL, brown, slightly moist, non cohesive, weathered, decomposing rock fractured to 3" gravel size by bucket, hard drilling.
13.0'	SP-SM			NP	71	9	POORLY GRADED SAND WITH SILT AND GRAVEL, brown, slightly moist, non cohesive, weathered rock and cobbles to 8", white, grey and rust brown, fractures and breaks apart, drilling stopped due to rock and cobbles.

CBH 86-23

DEPTH	LOG	MC	LL	PI	-4	-200	
	SM		28	3	96	18	SILTY SAND, brown, moist, dense to very dense, non cohesive.
				NP	92	13	light drilling, gravel to 1".
				NP	90	13	
				NP	94	15	darker brown, gravel to 2".
				NP	77	13	SILTY SAND WITH GRAVEL, brown, moist, dense, non cohesive, whitish grey, weathered cobbles disintergrate with hand pressure
			23	1	96	17	SILTY SAND, brown, moist, dense to very dense, non cohesive, few subangular cobbles.
22.0'			25	1	93	20	increase in moisture, gravel to 1".
	SC-SM		26	6	93	18	SILTY, CLAYEY SAND; brown, moist, dense, some cobbles and weathered rock, gravel to 1/2".
29.0'			24	4	95	22	
	SM			NP	91	17	SILTY SAND, brown, moist, some weathered rock, gravel to 1".
				NP	87	13	
				NP	91	13	increase in weathered rock.
			22	2	95	19	
44.0'				NP	92	16	
47.0'	SW-SM	7		NP	81	11	WELL GRADED SAND WITH SILT AND GRAVEL, brown, moist, gravel to 1-1/2", highly weathered rock.
	SM			NP	88	13	SILTY SAND; brown, moist, weathered rock, gravel to 1"
			25	1	85	16	SILTY SAND WITH GRAVEL, brown, moist, weathered rock, gravel to 3".
				NP	87	13	SILTY SAND, brown, moist, weathered rock, gravel to 1".
60.0'				NP	86	15	drilling stopped due to rocks.

SCALE 1 IN = 5 FT

SCALE 5 0 5 10 15 FEET

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

IMPERVIOUS BORROW AREA
SOIL LOGS
CBH 86 17 THRU CBH 86-23

SUBMITTED BY: DATE APPROVED:

SPEC. NO. DACW 09- ____ 8- ____

DISTRICT FILE NO.

SHEET

SAFETY PAYS

PLATE A-67

CBH 86-24

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
			31	7	96	31	SILTY SAND; brown, moist, slightly cohesive, hard drilling.
	SM			NP	95	17	light brown, non cohesive, gravel to 1 1/2", decomposed rock.
8.5'			26	2	98	27	cobbles to 4", hard drilling, more decomposed rock.
	SW-SM		22	3	91	12	WELL GRADED SAND WITH SILT; dark brown, increase in moisture.
11.5'				NP	76	13	SILTY SAND WITH GRAVEL; light brown, moist, cobbles to 5", more decomposed rock.
	SM	6	25	1	84	20	slightly cohesive, hard drilling.
21.0'			25	1	93	22	SILTY SAND; light brown, moist, slightly cohesive, gravel to 2", hard drilling.
	SC-SM		22	4	98	19	SILTY CLAYEY SAND; light brown, less moisture, slightly cohesive.
24.0'	SM			NP	99	30	SILTY SAND; dark brown, increase in moisture, more cohesive.
26.5'				NP	63	13	SILTY SAND WITH GRAVEL; brown, moist, subangular decomposed cobbles, hard drilling stopped due to rocks.

CBH 86-26

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
	SC		31	10	99	45	CLAYEY SAND; dark brown, moist, cohesive.
1.5'	SM		38	7	100	47	SILTY SAND; dark brown, moist, very dense, well cemented.
2.0'	SC		33	11	98	30	CLAYEY SAND; light brown, moist, slightly cohesive.
4.5'	SM	10	26	3	99	31	SILTY SAND; dark brown, moist, slightly cohesive.
9.0'	SC-SM		28	6	99	28	SILTY, CLAYEY SAND; dark brown, moist, slightly cohesive.
12.0'			29	3	98	38	SILTY SAND; dark brown, moist, small amount of decomposed rock.
	SM		29	2	98	33	
18.0'	SP-SM			NP	86	9	POORLY GRADED SAND WITH SILT; light brown, moist, large amount of decomposed rock, hard drilling.
20.0'	SM			NP	97	16	SILTY SAND; light brown, moist, less weathered rock and cobbles.
25.0'	SC		22	9	97	20	CLAYEY SAND; light brown, moist, decomposed rock.
27.5'	SC-SM		26	6	98	37	SILTY, CLAYEY SAND; light brown, moist, decomposed rock
30.0'	SM			NP	93	14	SILTY SAND; light brown, moist, decomposed rock.
33.0'	SC-SM	10	26	6	97	28	SILTY, CLAYEY SAND; light brown, moist, cohesive.
36.0'	SM		25	1	98	24	SILTY SAND; light brown, moist.
38.0'	NO	SAMPLE	TAKEN				drilling stopped due to large rocks.
40.0'							

CBH 86-25

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
			32	5	99	36	SILTY SAND; red-brown, moist, cohesive easy drilling.
	SM		19	1	98	28	light red-brown, slightly cohesive, harder drilling.
7.0'			21	1	98	18	
	SW-SM			NP	95	10	WELL GRADED SAND WITH SILT; light red-brown, moist, weathered, decomposed rock.
13.0'				NP	93	10	gravel to 1".
	SM		23	1	97	28	SILTY SAND; brown, moist, slightly cohesive, less decomposed rock, very dense.
16.5'			23	1	75	19	SILTY SAND WITH GRAVEL; brown, moist, decomposed rock, gravel to 1 1/2". cobbles to 6" drilling stopped due to large rock.

CBH 86-27

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
			28	1	92	17	SILTY SAND; dark brown, moist, gravel to 1/2"
	SM			NP	92	16	light brown, dry.
4.0'							
	SP-SM			NP	90	9	POORLY GRADED SAND WITH SILT, light brown, dry.
7.0'			22	1	77	5	SILTY SAND WITH GRAVEL, light brown, dry, gravel to 1 1/2"
				NP	93	16	gravel to 2", decomposed rock.
	SM		23	1	89	18	SILTY SAND; light brown, damp, gravel to 1"
			24	2	90	22	red-brown, moist.
19.0'				NP	68	17	SILTY SAND WITH GRAVEL, red-brown, moist, gravel to 3", cobbles to 5". drilling stopped due to large rocks.

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

CBH 86-28

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
	SM			NP	75	14	SILTY SAND WITH GRAVEL; light brown, damp, few small cobbles, decomposed rock.
3.0'	SW-SM			NP	70	11	WELL GRADED SAND WITH SILT AND GRAVEL; light brown, damp, gravel to 3" decomposed rock.
7.5'				NP	82	12	gravel to 3/4"
			29	1	91	23	SILTY SAND; red-brown, moist, slightly cohesive gravel to 3/4".
			32	1	92	40	light red-brown, moist, small cobble and decomposed rock.
			27	1	93	15	
			25	2	87	29	
				NP	79	16	SILTY SAND WITH GRAVEL; light red-brown, moist, gravel to 1 1/2", some decomposed rock, hard drilling.
	SM		23	1	80	21	
				NP	76	13	dark brown, more moisture, gravel to 2".
			22	3	86	20	
			31	7	74	20	slight cohesion, gravel to 3".
			27	1	67	16	small cobbles.
40.0'				NP	73	24	

CBH 86-29

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
			24	1	97	25	SILTY SAND, dark brown, moist.
				NP	97	19	light brown, dry, small cobbles and decomposed rocks.
	SM		27	2	99	24	
10.0'			29	5	100	36	clods, few small cobbles, hard drilling.
13.0'	SC		31	11	100	27	CLAYEY SAND; light brown, damp, weathered, decomposed rocks.
			27	1	99	25	SILTY SAND; light brown, damp, weathered, decomposed rocks.
			26	1	97	22	
				NP	93	14	
				NP	92	13	few cobbles to 4".
				NP	89	13	
				NP	96	20	moist, slight cohesion.
	SM	7		NP	93	25	
			25	1	94	26	brown, clods.
		8	27	3	100	33	
			26	1	72	15	SILTY SAND WITH GRAVEL; brown, damp, to 1 1/2", cobbles to 4".
				NP	88	17	SILTY SAND; light brown, moist, slightly cohesive, decomposed rock, few small cobbles.
			24	1	90	15	red-brown, medium dense.
50.0'				NP	86	15	

CBH 86-30

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
	SC-SM		23	4	100	34	SILTY, CLAYEY SAND; brown, damp, medium dense, slightly cohesive.
3.0'	SC		29	9	99	31	CLAYEY SAND; brown, damp, medium dense, cohesive.
6.0'			28	4	100	26	SILTY SAND; brown, damp, medium dense.
			27	2	100	48	slightly cohesive.
			21	3	92	18	gravel to 1".
			24	2	96	26	gravel to 2", decomposed rock.
	SM		24	2	99	16	
			29	1	99	17	
		9	23	1	95	38	dark brown, damp to moist, clods.
		11	29	6	99	27	brown, damp, cohesive.
			28	3	98	30	slightly cohesive.
36.0'			24	2	98	29	
38.0'	SC		23	8	98	27	CLAYEY SAND; brown, damp, medium dense, cohesive.
40.0'	SM		24	2	95	22	SILTY SAND; brown, damp, medium dense, slight cohesion.
44.0'	SC-SM		25	5	88	17	SILTY, CLAYEY SAND; brown, medium dense, slightly cohesive, subrounded gravel to 3", cobbles to 4".
46.0'	SW-SM			NP	90	11	WELL GRADED SAND WITH SILT; brown, damp, medium dense, subrounded gravel to 3".
50.0'	SM		21	3	95	21	SILTY SAND; brown, damp, medium dense, subrounded gravel to 2".

SCALE: 1 IN = 5 FT.

SCALE 5 0 5 10 15 FEET

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY: F. Castellanos
CHECKED BY:

SANTA ANA RIVER, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

IMPERVIOUS BORROW AREA
SOIL LOGS
CBH 86-24 THRU CBH 86-30

SUBMITTED BY: DATE APPROVED:
CHIEF, BRANCH

SPEC. NO. DACW09-____-B-____
DISTRICT FILE NO.

SHEET

PLATE A-68

CBH 86-31

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
7.0'	SM	NO SAMPLE TAKEN					SILTY SAND, brown, damp, many grass roots.
4.0'	ML		23	3	100	57	SANDY SILT, brown, damp, medium dense, cohesive, no roots.
5.5'	SM		28	2	100	48	SILTY SAND, brown, damp, dense, cohesive.
7.0'	SC	9	33	10	100	47	CLAYEY SAND, lighter brown, damp, medium dense.
	SM		28	1	100	35	SILTY SAND, brown, damp, medium dense, slightly cohesive.
15.0'			31	3	98	39	
18.0'	SC		29	8	97	18	CLAYEY SAND, lighter brown, damp, medium dense.
22.0'	SM		27	4	100	42	SILTY SAND, brown, damp, medium dense, slightly cohesive.
25.0'	SC-SM		27	6	96	33	SILTY, CLAYEY SAND, brown, damp, medium dense, cohesive.
	SM		27	3	93	25	SILTY SAND, brown, damp, slightly cohesive.
			29	5	96	22	gravel to 1"
			22	1	94	14	medium dense.
36.5'			22	3	91	18	
38.0'	SW-SM			NP	96	12	WELL GRADED SAND WITH SILT, lighter brown, damp, medium dense, subrounded gravel to 1"
	SM			NP	87	16	SILTY SAND, brown, damp, slightly cohesive, subrounded gravel to 3", occasional subrounded cobble.
45.0'			21	1	90	16	gravel to 2".

CBH 86-34

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
			24	1	100	35	SILTY SAND; dark brown, moist, slightly cohesive, few organics.
	SM		23	1	100	49	light brown, clods, decomposed rock, due to large rock moved drillrig 5' to new location.
		8	25	3	97	26	began logging again at 5'.
11.5'			24	1	97	30	few cobbles.
15.5'	SW-SM			NP	89	5	WELL GRADED SAND WITH SILT, light brown moist, gravel to 3"
19.0'	SC	11	24	10	100	47	CLAYEY SAND; dark brown, moist, cohesive, large clods, decomposed rock.
	SC-SM		28	6	99	48	SILTY, CLAYEY SAND, dark brown, moist, cohesive, large clods, gravel to 1/2".
			21	7	99	42	
27.5'			27	7	97	41	gravel to 1".
29.0'	SM		29	4	97	43	SILTY SAND, dark brown, moist, cohesive, clods, gravel to 1".
32.0'	SC-SM		28	6	97	38	SILTY, CLAYEY SAND, dark brown, moist, cohesive, clods.
35.0'	SM		27	4	97	36	SILTY SAND; red-brown, moist, gravel to 1/2", 1" to 2" lenses of decomposed rock.
37.0'	SC-SM		24	7	99	36	SILTY, CLAYEY SAND, red-brown, moist, few 3" rock fragments.
38.0'	SM		24	1	96	33	SILTY SAND, red-brown, moist, rock fragments; drilling stopped due to large rock.

DEPTH	LOG
2.0'	SM
	SW-SM
9.5'	
	SM
17.0'	
21.0'	SP-SM
23.5'	SC-SM
27.0'	SM
30.0'	SC-SM
34.0'	SM
36.0'	SP-SM
40.0'	SM

CBH 86-32

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
4.0'	SC		36	12	99	41	CLAYEY SAND, brown, moist, cohesive.
	SM		25	2	90	20	SILTY SAND, brown, dry, cobbles to 6", clods.
		8	26	2	99	42	brown, moist, slight cohesion, few cobbles, rock at 11', moved drill rig 10' to new location.
			28	2	100	37	began logging again at 11', clods, decomposed rock.
			28	1	100	33	
			26	4	89	16	SILTY SAND WITH GRAVEL, brown, moist, cobbles to 5", hard drilling, bucket fracturing rock into gravel and cobble size material.
				NP	73	13	
23.0'							cobbles to 8", decomposed rock, drilling stopped due to large rock.

CBH 86-33

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
	SM		34	6	99	29	SILTY SAND, red-brown, moist, cohesive, large clods.
5.0'			31	2	100	30	light brown, dry, dense, cemented, small clods, hard drilling
7.0'	SC-SM	6	25	5	98	27	SILTY, CLAYEY SAND, light brown, dry, dense, cemented, small clods.
11.0'	SP-SM	2		NP	53	5	POORLY GRADED SAND WITH SILT AND GRAVEL, light brown, dry, gravel to 2", cobbles to 5", bucket fracturing rock into cobble sizes.
	SW-SM			NP	79	8	WELL GRADED SAND WITH SILT AND GRAVEL, light brown, dry, dense, gravel to 3", subrounded cobbles to 9" decomposed rock, hard drilling.
15.0'		2		NP	66	8	gravel to 2", bucket fracturing large rock into gravel, drilling stopped due to rock.

DEPTH	LOG
3.0'	SW-SM
5.5'	SM
9.0'	SC
11.0'	SP-SM
19.0'	SM
23.0'	GP-GM
30.0'	SM

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

CBH 86-35

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
2.0'	SM		25	1	89	17	SILTY SAND; light brown, moist, few organics.
9.5'	SW-SM			NP	86	12	WELL GRADED SAND WITH SILT; beige, dry, few decomposed rocks, easy drilling.
				NP	87	9	WELL GRADED SAND WITH SILT AND GRAVEL; beige, dry, gravel to 3", decomposed rock, easy drilling.
				NP	84	10	
17.0'	SM		26	1	87	15	SILTY SAND; red-brown, moist, gravel to 2".
		8	25	2	77	19	SILTY SAND WITH GRAVEL; red-brown, moist, few cobbles to 5".
21.0'	SP-SM		25	1	62	12	POORLY GRADED SAND WITH SILT AND GRAVEL; red-brown, moist, few cobbles to 5".
23.5'	SC-SM		25	7	78	17	SILTY, CLAYEY SAND WITH GRAVEL; red-brown, moist, cohesive clods, decomposed rock.
27.0'	SM		28	4	72	14	SILTY SAND WITH GRAVEL; red-brown, moist, slightly cohesive, gravel to 3", weathered decomposed rock.
30.0'	SC-SM		25	7	89	14	SILTY, CLAYEY SAND; red-brown, moist, cohesive, gravel to 3/4", few small clods, weathered decomposed rock, easy drilling.
34.0'	SM		23	2	91	16	SILTY SAND; red-brown, moist, weathered decomposed rock, easy drilling.
36.0'	SP-SM			NP	76	12	POORLY GRADED SAND WITH SILT AND GRAVEL; red-brown, moist, medium dense to dense, weathered rock, bucket fracturing rock into gravel sizes to 3".
40.0'	SM			NP	83	14	SILTY SAND WITH GRAVEL; red-brown, dense, slightly cohesive, small clods, weathered decomposed rock.

CBH 86-37

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
3.5'	SC		29	9	98	27	CLAYEY SAND; moist, cohesive, small clods, some decomposed rock.
10.0'	SP-SM			NP	83	9	POORLY GRADED SAND WITH SILT AND GRAVEL, moist, many cobbles to 8", weathered decomposed rock, hard drilling.
				NP	83	6	very hard digging and caving.
							drilling stopped due to large rocks.

CBH 86-38

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
4.0'	SM			NP	93	16	SILTY SAND; dark brown, moist, few cobbles, decomposed rock, organics.
12.0'	SW-SM			NP	95	8	WELL GRADED SAND WITH SILT; light brown, moist, noncohesive.
				NP	84	7	WELL GRADED SAND WITH SILT AND GRAVEL; dark brown, moist, slight cohesion, gravel to 1", few cobbles, weathered rock, caving from 8 foot to 12 foot depth.
21.5'	SM			NP	93	15	SILTY SAND; dark brown, moist, slight cohesion, small clods, gravel to 1".
			21	1	97	16	
			20	2	96	15	
28.0'	SW-SM			NP	97	7	WELL GRADED SAND WITH SILT; light brown, moist, gravel to 1-1/2".
				NP	92	10	

CHB 86-36

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
3.0'	SW-SM			NP	80	12	WELL GRADED SAND WITH SILT AND GRAVEL; red-brown, damp, weathered decomposed rock.
5.5'	SM			NP	93	17	SILTY SAND; red-brown, damp, gravel to 2", small clods, decomposed rock.
9.0'	SC		24	8	86	18	CLAYEY SAND; red-brown, damp, decomposed rock.
11.0'	SP-SM			NP	77	8	POORLY GRADED SAND WITH SILT AND GRAVEL; light brown, damp, dense, weathered decomposed rock, hard drilling.
19.0'	SM		21	2	87	16	SILTY SAND; red-brown, moist, weathered decomposed rock.
		8	24	2	88	17	cobbles to 5" easy drilling
23.0'	GP-GM			NP	40	7	POORLY GRADED GRAVEL WITH SILT AND SAND; light brown, moist, slightly cohesive, gravel to 3", weathered decomposed rock, easy drilling.
30.0'	SM			NP	83	14	SILTY SAND WITH GRAVEL; light brown, moist, weathered decomposed rock, gravel to 1-1/2".
				NP	80	13	red-brown, few cobbles, gravel to 3/4"
							drilling stopped due to large rock.

SCALE: IN. = 5 FT.

SCALE 5 0 5 10 15 FEET

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

IMPERVIOUS BORROW AREA
SOIL LOGS
CBH 86-31 THRU CBH 86-38

SUBMITTED BY:
DATE APPROVED:
SPEC. NO. DACW09-____-B-____
DISTRICT FILE NO.
SHEET

PLATE A-69

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

CBT 86-1

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
1.0'	SM		28	4	75	17	SILTY SAND WITH GRAVEL; brown, damp, few cobbles and boulders to 18".
2.5'	SC-SM		28	6	92	23	SILTY, CLAYEY SAND; red-brown, damp, 10% cobbles, occasional boulder to 18".
	NO	SAMPLE	TAKEN				Visual Estimate: 20% sand 30% gravel 20% cobbles 30% boulders to 15".
8.0'							

CBT 86-2

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
4.0'	SM		23	2	97	20	SILTY SAND, light brown, dry, moderate cohesion.
10.0'	SW-SM			NP	80	10	WELL GRADED SAND WITH SILT AND GRAVEL; light brown, dry, no cohesion, 4 % cobbles to 6".
				NP	78	12	
12.0'	SM		22	2	82	15	SILTY SAND WITH GRAVEL; light brown, dry, moderate cohesion, gravel to 2".
15.5'	SC-SM		23	5	80	18	SILTY, CLAYEY SAND WITH GRAVEL; light brown, dry, moderate cohesion, gravel to 2".

DEPTH	LOG	MC	LL	
3.0'	SM	12	24	
			22	
6.0'	SW-SM			
10.5'	SM			

CBT 86-5

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
			30	4	100	36	SILTY SAND; red-brown, damp, roots.
	SM		31	6	100	29	dense, moderately cemented.
				NP	100	27	light red-brown, slightly damp.
				NP	96	13	light brown, gravel to ½".
		NO	SAMPLE	TAKEN			gravel to 1½".
13.0'							
16.0'	SW-SM			NP	93	10	WELL GRADED SAND WITH SILT, light brown, slightly damp, dense, few cobbles to 7".

CBT 86-6

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
2.0'	SM		29	1	99	31	SILTY SAND; red-brown, damp, top soil.
3.0'	SW-SM		36	8	92	20	slightly damp.
6.0'	SP-SM			NP	84	11	WELL GRADED SAND WITH SILT AND GRAVEL; brown, dry, moderately cemented gravel to 2", gravel is fractured white rock
				NP	72	8	
10.0'	SW-SM			NP	94	11	POORLY GRADED SAND WITH SILT AND GRAVEL; brown, dry, gravel to 2", fractured decomposed rock.
				NP	92	7	WELL GRADED SAND WITH SILT, brown, dry, gravel to 1", few cobbles to 5".
14.0'	GW			NP	46	4	light brown, gravel to 2", white rock fragments.
							WELL GRADED GRAVEL WITH SAND, brown, dry, gravel to 3", appears to be rock fragments, weathered rock.

DEPTH	LOG	MC	LL
2.0'	SM		25
3.5'	SW-SM		
4.5'	SP		
6.0'	SM		
	SW-SM		
10.0'			
12.0'	SP-SM		
	SW-SM		
15.0'			

CBT 86-9

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
1.0'	SC-SM		23	6	64	36	CLAYEY, SILTY SAND; dark red-brown, damp,
	SC		31	15	77	21	gravel to 3".
3.5'			31	12	91	15	CLAYEY SAND WITH GRAVEL; dark red-brown,
4.5'	SP-SM			NP	73	11	dry, cemented, 10 % cobbles to 10".
7.0'	SW-SM			NP	62	5	CLAYEY SAND; dark red-brown, dry, cemented, 1-2½' boulder.
	SP-SM			NP	79	10	POORLY GRADED SAND WITH SILT AND GRAVEL, red-brown, dry, gravel to 3", 5 % cobbles to 12".
10.0'							
12.0'	SW-SM			NP	76	6	WELL GRADED SAND WITH SILT AND GRAVEL; light red-brown, dry, gravel and cobbles to 4", decomposed rock.
							POORLY GRADED SAND WITH SILT AND GRAVEL; light red-brown, gravel and cobbles to 12", layer of cobbles from 7 foot to 12 foot depth.
							WELL GRADED SAND WITH SILT AND GRAVEL; light red-brown, gravel and cobbles to 12", decompesed rock, tronching stopped due to large rock.

CBT 86-10

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
0.5'	NO	SAMPLE		TAKEN			TOP SOIL, grass roots.
1.5'	SC		28	11	98	28	CLAYEY SAND; red-brown, damp, cemented clods, gravel to 3".
4.0'	SW-SM			NP	85	11	
				NP	92	6	WELL GRADED SAND WITH SILT AND GRAVEL; orange-brown, damp, slightly cemented clods, gravel to 3".
	SP-SM			NP	82	12	POORLY GRADED SAND WITH SILT; light orange-brown, damp, medium dense, gravel to 3", 12" cobble at 7'.
10.0'							
12.0'	SW-SM			NP	78	8	POORLY GRADED SAND WITH SILT AND GRAVEL; light red—brown, dense, gravel to 2"
14.0'	SM			NP	96	16	WELL GRADED SAND WITH SILT AND GRAVEL; light red-brown, lightly cemented clods.
16.0'	SW-SM			NP	97	12	SILTY SAND, orange-brown, gravel to 1"
							WELL GRADED SAND WITH SILT, red-brown, gravel to 3".

DEPTH	LOG	MC	LL
			22
4.0'	SC	5	27
6.0'	SC-SM		27
8.0'	SW-SM		
	SP-SM		
11.0'			
12.0'	SW		

CBT 86-12

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
1.0'	SC-SM		25	4	100	43	SILTY, CLAYEY SAND; dark brown, moist, top soil.
	SM	6	30	2	100	42	SILTY SAND; red-brown, white specks, dry, very dense, well cemented gravel to ½".
6.0'			30	5	100	35	
8.0'	SC		31	9	100	28	CLAYEY SAND; brown, dry, very dense, cemented, gravel to 2".
10.0'	SC-SM		24	5	95	20	SILTY, CLAYEY SAND; brown, damp, dense, gravel to ¾".
13.0'	SM		22	1	89	15	SILTY SAND; brown, damp, gravel to 1", few cobbles to 12", tronching stopped due to cobbles and boulders.

IPTION

t brown, dry, moderate

WITH SILT AND GRAVEL, no cohesion, 4 % cobbles

GRAVEL, light brown, dry, to gravel to 2".

ND WITH GRAVEL, light brown, hesion, gravel to 2".

CBT 86-3

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
	SM	12	24	3	99	25	SILTY SAND; red-brown, moist, dense, gravel to 3/4"
3.0'			22	1	98	35	light red-brown, dry, moderately cemented.
6.0'	SW-SM			NP	96	12	WELL GRADED SAND WITH SILT; brown, dry, very dense, gravel to 1".
	SM			NP	93	19	SILTY SAND; brown, dry, dense, gravel to 2".
10.5'							

CBT 86-4

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
			23	2	92	22	SILTY SAND; red-brown, damp, gravel to 1"
	SM		24	2	96	20	light red-brown, dense.
7.0'			24	3	91	13	brown.
	SP-SM			NP	73	11	POORLY GRADED SAND WITH SILT AND GRAVEL; brown, dry, dense, gravel to 1 1/2".
8.0'							
	SM		23	1	89	23	SILTY SAND; red-brown, damp, dense, slightly cohesive, gravel to 1 1/2"
16.0'		7		NP	88	20	
17.0'	SW-SM			NP	87	11	WELL GRADED SAND WITH SILT; red-brown, damp, dense, gravel to 1 1/2".

IPTION

-brown, damp, top soil

ND WITH SILT AND GRAVEL, rately cemented gravel to 2", red white rock.

SAND WITH SILT AND GRAVEL, el to 2", fractured decomposed

ND WITH SILT; brown, dry, gravel se to 2"

ravel to 2" white rock fragments.

AVEL WITH SAND; brown, dry, gravel to be rock fragments, weathered

CBT 86-7

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
2.0'	SM		25	1	97	18	SILTY SAND; light red-brown, damp, gravel to 1/2" fragments of decomposed rock.
3.5'	SW-SM			NP	97	11	
4.5'	SP			NP	97	3	WELL GRADED SAND WITH SILT; red-brown, dry, gravel to 1", fragments of decomposed rock.
6.0'	SM			NP	98	13	POORLY GRADED SAND; red-brown, dry, gravel to 1", fragments of decomposed rock.
	SW-SM			NP	82	6	
10.0'				NP	90	9	SILTY SAND; red-brown, dry, angular gravel to 2", fragments of decomposed rock.
12.0'	SP-SM			NP	82	7	WELL GRADED SAND WITH SILT AND GRAVEL; light brown, dry, fragments of decomposed rock to 3".
	SW-SM			NP	70	6	
15.0'				NP	75	9	gravel to 2".
							POORLY GRADED SAND WITH SILT AND GRAVEL; brown, dry, rock fragments.
							WELL GRADED SAND WITH SILT AND GRAVEL; light brown, fewer rock fragments.
							brown, gravel to 2".

CBT 86-8

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
1.5'	SC		31	10	100	39	CLAYEY SAND; dark red-brown, damp, cohesive, gravel to 3/8", many roots.
			28	4	100	17	
			25	2	100	24	SILTY SAND; red-brown, dry, slightly cemented, gravel to 3/8".
	SM			NP	93	15	light red-brown, gravel to 1/2", lightly cemented clods of decomposed rock.
				NP	96	13	gravel to 3", lightly cemented decomposed rock to 4".
				NP	87	13	SILTY SAND WITH GRAVEL; light brown, dry, gravel to 3", lightly cemented decomposed rock to 4".
				NP	96	22	
15.0'				NP	97	14	SILTY SAND; light brown, dry, few fragments of decomposed rock to 2" gravel to 1".

SCRIPTION

L roots.

red-brown, damp, cemented to 3".

ND WITH SILT AND GRAVEL, damp, slightly cemented to 3"

SAND WITH SILT; light damp, medium dense, gravel at 1".

SAND WITH SILT AND GRAVEL, wn, dense, gravel to 2"

ND WITH SILT AND GRAVEL, m, lightly cemented clods, range-brown, gravel to 1".

ND WITH SILT; red-brown,

CBT 86-11

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
			22	8	96	30	CLAYEY SAND; orange-brown, grass, roots, top soil.
	SC	5	27	11	100	34	red-brown, dry, very dense, well cemented, cobbles to 9".
4.0'							
6.0'	SC-SM		27	6	86	15	SILTY, CLAYEY SAND WITH GRAVEL; orange-brown, gravel to 3", bucket grinding rock into many fragments.
8.0'	SW-SM			NP	90	7	
	SP-SM			NP	63	9	WELL GRADED SAND WITH SILT; brown, dry, gravel to 1/4", many cobbles and few boulders to 5", rock fragments.
11.0'							
12.0'	SW			NP	70	3	POORLY GRADED SAND WITH SILT AND GRAVEL; light brown, dry, gravel to 3", 20 % cobbles to 9", few boulders to 1 1/2".
							WELL GRADED SAND WITH GRAVEL; brown and grey, dry, gravel to 3/4", rock fragments

SCALE: 1 IN. = 5 FT.

SCALE 5 0 5 10 15 FEET

SYMBOL	DESCRIPTION	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

IMPERVIOUS BORROW AREA
SOIL LOGS
CBT 86-1 THRU CBT 86-12

SUBMITTED BY:
DATE APPROVED:
CHIEF, DIVISION
SPEC. NO. DACW09- - -
DISTRICT FILE NO.
SHEET

PLATE A-78

CBT 86-13

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
			22	2	97	38	SILTY SAND, dark brown, moist, gravel to 3/4"
	SM		22	2	99	42	brown, damp, cemented, very dense, gravel to 2", few cobbles to 12".
5.0'			27	5	100	41	
7.0'	SC-SM		27	6	99	32	SILTY, CLAYEY SAND; brown, dry, cemented, gravel to 1", few cobble fragments to 6"
	SM		24	3	75	13	SILTY SAND WITH GRAVEL; brown, dry, gravel to 3".
12.0'			24	3	96	13	SILTY SAND, brown, damp, gravel to 5/8"
13.0'	SC-SM		28	6	97	19	SILTY CLAYEY SAND, brown, damp, gravel to 1".
	SM		25	2	98	20	SILTY, SAND, brown, damp, slightly cohesive, clods.
17.0'			25	1	97	20	gravel to 1".

CBT 86-14

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
			27	3	92	19	SILTY SAND; dark brown, moist, gravel to 5/8"
	SM		29	5	87	20	brown, damp, cemented clods, gravel to 2"
5.5'				NP	88	13	few cobbles to 12"
8.0'	SP-SM			NP	70	7	POORLY GRADED SAND WITH SILT AND GRAVEL; brown, dry, gravel to 2", few cobbles to 5"
10.0'	SP			NP	87	4	POORLY GRADED SAND, brown, dry, gravel to 3", few boulders to 1 1/2"
12.0'	SW-SM			NP	86	11	WELL GRADED SAND WITH SILT; brown, dry, gravel to 1", few clods.
12.5'	SM		24	5	95	25	SILTY SAND; brown, dry, many cemented clods
14.0'	SP-SM			NP	84	7	gravel to 3/4"
16.0'	SW-SM			NP	86	8	POORLY GRADED SAND WITH SILT AND GRAVEL; brown, dry, gravel and cobbles to 5"
							WELL GRADED SAND WITH SILT; brown, dry, gravel to 2"

CBT 86-16

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
0.5'	SM		28	1	100	36	SILTY SAND, dark brown, moist, gravel to
		13	30	12	100	37	3/8", top soil.
	SC		35	11	100	28	CLAYEY SAND; red-brown, moist.
4.0'							light red-brown, dry, dense, cemented,
	SM			NP	90	16	gravel to 2".
7.0'							SILTY SAND; light brown, dry, cemented,
9.0'	SW-SM			NP	97	9	gravel to 1/2"
	SM			NP	91	15	WELL GRADED SAND WITH SILT; light brown,
11.0'							dry, gravel to 1".
	GP-GM			NP	34	6	SILTY SAND; light brown, dry, moderately cemented clods, gravel to 1"
14.0'							POORLY GRADED GRAVEL WITH SILT AND SAND, light brown, dry, 20 % cobbles to 5", few subrounded and subangular cobbles to 12" at 12 feet depth.

CBT 86-17

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
1.0'	SC	11	26	8	85	22	CLAYEY SAND, dark red-brown, moist, gravel
2.0'	SM	4	24	3	98	16	3/8", roots.
	SC-SM		24	5	93	14	SILTY, SAND; light red-brown, slightly damp,
5.0'							dense, gravel to 3", decomposed rock.
6.0'	SW-SM		24	2	95	12	SILTY CLAYEY SAND; light red-brown, slightly
7.5'	SC	8	22	8	86	16	damp, very dense, decomposed rock.
9.0'	SW-SM			NP	93	12	WELL GRADED SAND WITH SILT; light brown,
							dense, gravel to 1 1/2", gravel breaks into smaller
	SM			NP	95	18	sizes, decomposed rock.
11.5'			20	2	89	16	CLAYEY SAND, red-brown, damp, dense, cemented
							WELL GRADED SAND WITH SILT, red-brown, damp, gravel to 2", gravel breaks into smaller sizes.
							SILTY SAND, red-brown, damp, gravel to 3", few cobbles to 5", boulders to 5' at bottom of trenc

CBT 86-19

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
1.5'	SC-SM	16	27	7	81	25	SILTY, CLAYEY SAND WITH GRAVEL, dark red-brown, moist, gravel to 3/4"
3.0'	SC	7	28	8	98	23	CLAYEY SAND, light red-brown, damp, dense, cemented, gravel to 1".
	SW-SM	6		NP	82	11	WELL GRADED SAND WITH SILT AND GRAVEL,
7.0'							light orange-brown, slightly damp, dense, moderately cemented, gravel to 3".
9.0'	SM			NP	87	16	SILTY SAND, light orange-brown, slightly,
11.0'	SW-SM			NP	86	6	damp, dense, moderately cemented, gravel to 3", cobbles and boulders to 14"
							WELL GRADED SAND WITH SILT, brown, slightly, damp, dense, fractured cobbles and boulders to 14".

CBT 86-20

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
1.5'	SM	12	22	1	100	36	SILTY SAND, dark brown, moist, gravel to
2.5'	SC	7	32	14	100	35	3/8", roots.
4.0'	SC-SM	4	23	5	96	16	CLAYEY SAND, dark red-brown, damp, very dense, cemented, gravel to 3/8"
				NP	97	17	SILTY, CLAYEY SAND, light brown, damp,
	SM						dense, lightly cemented, gravel to 1"
				NP	93	13	SILTY SAND, light brown, damp, dense,
							gravel to 2".
				NP	94	16	brown, gravel to 3", cobbles to 5"
10.0'							
				NP	96	12	red-brown, damp, subangular to subrounded gravel and cobbles to 4".
	SW-SM			NP	84	12	WELL GRADED SAND WITH SILT, brown, damp, dense, subangular to subrounded gravel to 1".
16.0'				NP	93	6	WELL GRADED SAND WITH SILT AND GRAVEL, brown, damp, dense, subangular to subrounded gravel and cobbles to 5".
							WELL GRADED SAND WITH SILT, brown, damp, subangular to subrounded gravel to 2"

CBT 86-22

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
			29	1	98	21	SILTY SAND; brown, moist, gravel to 2", roots
3.0'	SM		26	2	99	24	light brown, damp, dense, subrounded gravel to 2", occasional boulders to 2".
				NP	96	12	WELL GRADED SAND WITH SILT, light brown,
	SW-SM						damp, dense, cemented, gravel to 2", occasional decomposed rock fragments to 6"
9.0'				NP	94	11	
			24	2	97	19	SILTY SAND; red-brown, damp, moderately cemented, subangular to subrounded gravel to 1", occasional decomposed rock fragments to 6", 1-2' boulder.
	SM						
14.0'				NP	94	13	
15.5'	SW-SM			NP	92	5	WELL GRADED SAND WITH SILT, brown, damp, dense, subrounded gravel to 1", occasional decomposed rock fragments to 6"

CBT 86-23

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
1.0'	SC-SM		26	5	95	30	SILTY CLAYEY SAND, dark brown, moist,
3.5'	SM	7	24	3	96	21	gravel to 3/4".
							SILTY SAND, brown, damp, gravel to 1"
	SP-SM			NP	88	6	POORLY GRADED SAND WITH SILT AND GRAVEL, light brown, dry, gravel to 3", subrounded cobbles and boulders to 26"
				NP	75	7	
9.0'							
12.5'	SP			NP	82	3	POORLY GRADED SAND WITH GRAVEL; light brown, dry, gravel to 3", subrounded cobbles and boulders to 26", caving due to removal of cobbles and boulders.

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

CBT 86-14

LOG	MC	LL	PI	-4	-200	DESCRIPTION
		27	3	92	19	SILTY SAND, dark brown, moist, gravel to 5/8"
SM		29	5	87	20	brown, damp, cemented clods, gravel to 2"
		NP	88	13		few cobbles to 12"
SP-SM		NP	70	7		POORLY GRADED SAND WITH SILT AND GRAVEL; brown, dry, gravel to 2", few cobbles to 5"
SP		NP	87	4		POORLY GRADED SAND, brown, dry, gravel to 3", few boulders to 11/2"
SW-SM		NP	86	11		WELL GRADED SAND WITH SILT; brown, dry, gravel to 1", few clods.
SM	24	3	95	23		SILTY SAND, brown, dry, many cemented clods gravel to 3/8"
SP-SM		NP	84	7		POORLY GRADED SAND WITH SILT AND GRAVEL; brown, dry, gravel and cobbles to 5"
SW-SM		NP	86	8		WELL GRADED SAND WITH SILT, brown, dry, gravel to 2".

CBT 86-15

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
2.0'	SM		33	8	100	31	SILTY SAND, red-brown, damp, gravel to 1/2"
			31	4	75	16	SILTY SAND WITH GRAVEL, light red-brown, very dense, gravel to 2"
5.0'	SW-SM			NP	100	8	WELL GRADED SAND WITH SILT; light red-brown, very dense, well cemented, gravel to 3"
6.0'	SC-SM		26	7	100	15	SILTY, CLAYEY SAND, light red-brown, very dense, well cemented.

CBT 86-17

LOG	MC	LL	PI	-4	-200	DESCRIPTION
SC	11	28	8	95	22	CLAYEY SAND, dark red-brown, moist, gravel 5/8", roots.
SM	4	24	3	98	16	
SC-SM		24	5	93	14	SILTY, SAND, light red-brown, slightly damp, dense, gravel to 3", decomposed rock.
SW-SM		24	2	95	12	SILTY CLAYEY SAND, light red-brown, slightly damp, very dense, decomposed rock.
SC	8	22	8	96	16	
SW-SM			NP	93	12	WELL GRADED SAND WITH SILT, light brown, dense, gravel to 1 1/2" gravel breaks into smaller sizes, decomposed rock.
SM			NP	95	18	
		20	2	89	16	CLAYEY SAND, red-brown, damp, dense, cemented.
						WELL GRADED SAND WITH SILT; red-brown, damp, gravel to 2", gravel breaks into smaller sizes.
						SILTY SAND, red-brown, damp, gravel to 3", few cobbles to 5", boulders to 5' at bottom of trench.

CBT 86-18

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
1.0'	SC-SM	14	24	6	87	38	SILTY, CLAYEY SAND, dark brown, moist, gravel to 3/4", top soil.
2.5'	SC	7	31	14	100	43	
	SW-SM			NP	64	11	CLAYEY SAND; dark red-brown, damp, dense, gravel to 3/4", a boulder to 5'.
5.0'				NP	60	11	
6.0'	SC		29	11	75	18	WELL GRADED SAND WITH SILT AND GRAVEL; dark red-brown, damp, very dense, gravel to 1", fragments of decomposed rock to 5", few cobbles to 6".
7.0'	SW-SM			NP	54	12	
							CLAYEY SAND WITH GRAVEL; brown, dry, dense, cemented, gravel to 3", cobbles to 10".
							WELL GRADED GRAVEL WITH SILT AND SAND; brown, dry, dense, cemented, gravel to 3", cobbles to 10", boulders in bottom of trench.

CBT 86-20

LOG	MC	LL	PI	-4	-200	DESCRIPTION
SM	12	22	1	100	38	SILTY SAND, dark brown, moist, gravel to 3/8", roots.
SC	7	32	14	100	35	CLAYEY SAND, dark red-brown, damp, very dense, cemented, gravel to 3/8".
SC-SM	4	23	5	96	16	
SM			NP	97	17	SILTY, CLAYEY SAND, light brown, damp, dense, lightly cemented, gravel to 1".
			NP	93	13	SILTY SAND, light brown, damp, dense, gravel to 2".
			NP	94	16	brown, gravel to 3", cobbles to 5".
			NP	96	12	red-brown, damp, subangular to subrounded gravel and cobbles to 4".
SW-SM			NP	84	12	WELL GRADED SAND WITH SILT, brown, damp, dense, subangular to subrounded gravel to 1".
			NP	93	6	
						WELL GRADED SAND WITH SILT AND GRAVEL, brown, damp, dense, subangular to subrounded gravel and cobbles to 5".
						WELL GRADED SAND WITH SILT, brown, damp, subangular to subrounded gravel to 2".

CBT 86-21

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
		12	27	2	99	37	SILTY SAND, dark red-brown, moist, roots.
	SM	6	28	5	69	17	SILTY SAND WITH GRAVEL, light red-brown, damp, very dense, well cemented, gravel to 3/4"
4.0'							
	SC		27	8	98	21	CLAYEY SAND, light brown, damp, very dense, well cemented
6.5'				NP	95	8	
	SW-SM			NP	94	12	WELL GRADED SAND WITH SILT; light brown, slightly damp, dense, subangular to subrounded gravel to 3/4"
				NP	88	11	brown, gravel to 1", gravel to 3"
14.0'							

CBT 86-23

LOG	MC	LL	PI	-4	-200	DESCRIPTION
SC-SM		28	9	99	30	SILTY CLAYEY SAND, dark brown, moist, gravel to 3/4"
SM	7	24	3	98	21	SILTY SAND, brown, damp, gravel to 1".
SP-SM			NP	88	6	POORLY GRADED SAND WITH SILT AND GRAVEL, light brown, dry, gravel to 3", subrounded cobbles and boulders to 26"
			NP	75	7	
SP			NP	82	3	POORLY GRADED SAND WITH GRAVEL; light brown, dry, gravel to 3", subrounded cobbles and boulders to 26", caving due to removal of cobbles and boulders.

SCALE 1 IN = 5 FT.

SCALE 5 0 5 10 15 FEET

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

IMPERVIOUS BORROW AREA

SOIL LOGS

CBT 86-13 THRU CBT 86-23

SUBMITTED BY: DATE APPROVED:

SPEC. NO. DACW09-____-8-____

DISTRICT FILE NO.

SHEET

PLATE A-71

CBT 86 - 24

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
			23	2	97	23	SILTY SAND, dark brown, moist, sub-angular to subrounded gravel to 3"
3.0'	SM			NP	97	17	brown, damp, gravel to 2"
5.0'	SW-SM	4		NP	94	10	WELL GRADED SAND WITH SILT, brown, damp, dense, gravel to 3", few cobbles to 7"
8.0'	SM			NP	97	13	SILTY SAND, brown, damp, dense, gravel to 3".
9.0'	SW-SM			NP	90	11	WELL GRADED SAND WITH SILT, brown,
10.5'	SC-SM	13	28	6	100	48	damp, dense, gravel to 3/4".
	SP	NO	SAMPLE	TAKEN			SILTY, CLAYEY SAND, brown, damp, dense, gravel to 3/8".
14.0'		NO	SAMPLE	TAKEN			POORLY GRADED SAND, brown, damp, dense, subrounded cobbles and boulders to 18"
							POORLY GRADED SAND WITH GRAVEL, brown, damp, layer of gravel, cobbles and boulders in matrix of gravelly sand, gravel to 3", cobbles and boulders to 18"

CBT 86 - 25

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
0.5	SC-SM		20	4	97	38	SILTY, CLAYEY SAND, brown, moist,
3.0'	SM	12	34	8	98	32	top soil
		11					SILTY SAND, dark red-brown, damp, dense, gravel to 1"
6.0'	NO	SAMPLE	TAKEN				red-brown, gravel to 3", subrounded cobbles
							subrounded cobbles and boulders to 27"x 45"

CBT 86 - 27

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
1.5'	SM		21	3	100	40	SILTY SAND, brown, moist, roots.
	SC-SM		23	5	100	39	SILTY, CLAYEY SAND, red-brown, damp, dense, slightly cemented, gravel to 1"
5.0'			25	9	100	36	moderately cemented.
7.5'	SM		27	5	100	27	SILTY SAND, light brown, damp, moderately cemented, gravel to 3/8".
10.0'	SP-SM			NP	91	7	POORLY GRADED SAND WITH SILT, light brown, damp, dense, moderately cemented, gravel to 2"
13.0'	SM			NP	99	13	SILTY SAND, light brown, damp, dense, moderately cemented, gravel to 2"
17.0'	SW-SM			NP	98	8	WELL GRADED SAND WITH SILT, light brown, damp, dense, moderately cemented, few decomposed rock fragments to 3" at 16 foot depth

CBT 86 - 28

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
0.5'	SC-SM		23	6	100	38	SILTY CLAYEY SAND, red-brown, moist,
1.5'	SC		27	8	100	30	roots, top soil
			27	6	100	29	CLAYEY SAND, light red-brown, damp, moderately cemented, gravel to 3/8"
	SC-SM		25	5	100	27	SILTY, CLAYEY SAND, light red-brown, dry, very dense, cemented, gravel to 1/8"
10.0'			24	4	98	21	light brown, few subangular to sub-rounded cobbles to 6"
	SM			NP	97	13	moderately cemented, subangular to subrounded gravel to 3"
15.0'			22	1	98	19	SILTY SAND, light brown, dry, sub-rounded gravel to 3"
							subrounded gravel to 3/8" and cobbles to 5"

CBT 86 - 30

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
1.0'	SC		35	13	99	34	CLAYEY SAND, dark red-brown, moist, top soil
3.0'	SM	6	26	4	97	14	SILTY SAND, light red-brown, damp, very dense, gravel to 2"
5.0'	SC-SM		26	6	98	16	SILTY, CLAYEY SAND, light red-brown,
7.0'	SM	9	28	5	97	14	damp, very dense, occasional subround-ed boulder to 1".
	SW-SC		24	5	98	12	SILTY SAND, light red-brown, damp, dense, gravel to 1", occasional sub-rounded boulder to 1".
11.0'			25	8	95	11	
14.0'	SW-SM			NP	96	10	WELL GRADED SAND WITH CLAY; light red-brown, damp, dense, gravel to 3/4" gravel to 3"
							WELL GRADED SAND WITH SILT; light red-brown, damp, dense, gravel to 3"

CBT 86 - 31

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
0.5'	SC-SM		26	5	100	42	SILTY, CLAYEY SAND, dark brown, damp,
			42	24	100	26	top soil.
	SC	8	35	18	100	28	CLAYEY SAND, red-brown, moist, medium
6.0'		5	26	10	99	15	dense, cohesive. damp, very dense, moderately cemented gravel to 3/8"
7.0'	SC-SM		22	4	84	16	light brown, damp, dense, subrounded gravel to 2".
	SM		23	2	97	15	SILTY, CLAYEY SAND WITH GRAVEL, brown,
10.0'							damp, dense, gravel to 1", layer of
11.0'	SW-SM			NP	94	12	cobbles to 6".
	SP-SM			NP	92	11	SILTY SAND, brown, dry, dense, sub-rounded gravel and cobbles to 6"
14.0'							WELL GRADED SAND WITH SILT, brown,
	SM			NP	97	13	dry, dense, gravel and cobbles to 6"
17.0'							POORLY GRADED SAND WITH SILT, brown, damp, dense, gravel to 1".
							SILTY SAND, brown, damp, dense

CBT 86 - 33

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
			27	2	99	39	SILTY SAND, dark brown, moist,
	SM	6	23	1	99	38	slightly cohesive, gravel to 3/8"
5.0'							brown, damp, dense, lightly cemented, gravel to 2", few subangular to sub-rounded cobbles to 9"
8.0'	SC-SM	7	26	5	99	29	SILTY, CLAYEY SAND, brown, damp, dense, lightly cemented, gravel to 2"
	SM	7	23	3	95	18	SILTY SAND, brown, damp, dense, gravel to 3", few subrounded cobbles to 10"
12.0'							
	SW-SM		24	1	97	11	WELL GRADED SAND WITH SILT, brown, damp, dense, gravel to 1".
16.5'							

CBT 86 - 34

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
1.0'	SM		25	1	98	28	SILTY SAND, dark brown, moist, gravel to 1/2"
	SC-SM	10	25	4	98	47	SILTY, CLAYEY SAND, dark brown, moist, dense, subangular to subrounded grave to 1".
8.0'		12	27	5	98	32	gravel to 2".
	SM		28	3	98	43	SILTY SAND, dark brown, moist, dense,
11.0'							gravel to 2".
	SC	10	32	10	96	23	CLAYEY SAND, brown, moist, gravel to
14.0'							
16.5'	SM		23	1	98	23	SILTY SAND, brown, moist, gravel to 1

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

CBT 86-25

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
0.5	SC-SM		20	4	97	38	SILTY, CLAYEY SAND, brown, moist, top soil
1.0'	SM	12	34	8	98	32	SILTY SAND, dark red-brown, damp, dense, gravel to 1"
		11					
	NO SAMPLE TAKEN						red-brown, gravel to 3", subrounded cobbles.
							subrounded cobbles and boulders to 27"± 48"

CBT 86-26

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
0.5'	SC-SM	NO	SAMPLE	TAKEN			SILTY, CLAYEY SAND, brown, moist, top soil
	SM	NO	SAMPLE	TAKEN			
3.0		NO	SAMPLE	TAKEN			SILTY SAND, dark red-brown, damp, dense, cohesive, few gravel to 1"
	NO SAMPLE TAKEN						red-brown, few gravel to 3", subrounded cobbles.
6.0'							cobbles and boulders.

CBT 86-28

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
0.5	SC-SM		23	6	100	38	SILTY CLAYEY SAND, red-brown, moist, roots, top soil
1.5	SC		27	8	100	30	
			27	6	100	29	CLAYEY SAND, light red-brown, damp, moderately cemented, gravel to 3/8"
	SC-SM		25	5	100	27	SILTY, CLAYEY SAND, light red-brown, dry, very dense, cemented, gravel to 1/2"
			24	4	98	21	light brown, few subangular to subrounded cobbles to 6"
10.0'				NP	97	13	moderately cemented, subangular to subrounded gravel to 3"
	SM		22	1	98	19	SILTY SAND, light brown, dry, subrounded gravel to 3"
15.0'							subrounded gravel to 3/8" and cobbles to 5"

CBT 86-29

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
			23	1	59	41	SILTY SAND, dark brown, damp,
	SM		24	2	100	44	light red-brown, slightly damp, very dense, cemented,
7.5'			27	5	100	34	gravel to 3/8", subrounded cobbles to 8".
	SP						POORLY GRADED SAND WITH GRAVEL, light brown, damp, dense, gravel to 2". rock fragments.
15.0'		NO SAMPLE TAKEN					POORLY GRADED SAND, light brown, damp, dense, gravel to 1/2"

CBT 86-31

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
0.5	SC-SM		28	5	100	42	SILTY, CLAYEY SAND, dark brown, damp, top soil
			42	24	100	26	
	SC	8	35	18	100	28	CLAYEY SAND, red-brown, moist, medium dense, cohesive,
6.0'		9	26	10	99	15	damp, very dense, moderately cemented, gravel to 3/8"
7.0'	SC-SM		22	4	84	16	light brown, damp, dense, subrounded gravel to 2"
	SM		23	2	97	15	SILTY, CLAYEY SAND WITH GRAVEL, brown, damp, dense, gravel to 1", layer of cobbles to 6"
10.0'							
11.0'	SW-SM			NP	94	12	
14.0'	SP-SM			NP	92	11	SILTY SAND, brown, dry, dense, subrounded gravel and cobbles to 6".
	SM			NP	97	13	WELL GRADED SAND WITH SILT, brown, dry, dense, gravel and cobbles to 6"
17.0'							POORLY GRADED SAND WITH SILT, brown, damp, dense, gravel to 1".
							SILTY SAND, brown, damp, dense.

CBT 86-32

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
1.0'	SC-SM		24	4	99	46	SILTY, CLAYEY SAND, dark red-brown, damp, top soil
3.0'	SC	10	34	15	99	38	CLAYEY SAND, red-brown, damp, dense, gravel to 3/8"
	SM	7	28	3	98	18	SILTY SAND, brown, damp, dense, gravel to 3/8", boulder at 5 foot depth.
		6	25	3	98	27	
8.0'			25	3	99	29	gravel to 1", few cobbles to 3"
11.0'	SC-SM		25	4	99	20	gravel to 3", fragments of decomposed rock, 10x24" boulder at 8 foot depth.
	SW-SM			NP	78	11	SILTY, CLAYEY SAND, brown, damp, dense, gravel to 2", fragments of decomposed rock.
15.0'							WELL GRADED SAND WITH SILT AND GRAVEL, brown, damp, dense, subangular to subrounded gravel and cobbles to 5", decomposed rock.

CBT 86-34

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
1.0'	SM		25	1	98	28	SILTY SAND, dark brown, moist, gravel to 1/2"
	SC-SM	10	25	4	98	47	SILTY, CLAYEY SAND, dark brown, moist, dense, subangular to subrounded gravel to 1".
8.0'		12	27	5	98	32	gravel to 2".
11.0'	SM		28	3	98	43	SILTY SAND, dark brown, moist, dense, gravel to 2".
14.0'	SC	10	32	10	96	23	CLAYEY SAND, brown, moist, gravel to 1".
16.5'	SM		23	1	98	23	SILTY SAND, brown, moist, gravel to 1".

SCALE 1IN = 5 FT

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY: F. CASTELLANOS
CHECKED BY:

SANTA ANA RIVER, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

IMPERVIOUS BORROW AREA
SOIL LOGS
CBT 86-24 THRU CBT 86-34

SUBMITTED BY: DATE APPROVED:
CHIEF, ENGINEERING DIVISION

SPEC. NO. DACW09-____-B-____
DISTRICT FILE NO.

SHEET

PLATE A-72

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

CBT 87-1

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
	SM		22	2	[illegible]	27	SILTY SAND, brown, moist, fine to medium grained, some roots.
2.0'	SC-SM	[illegible]	[illegible]	6	99	43	SILTY CLAYEY SAND, brown, dry, dense, fine to medium grained.
5.0'	SM	8	28	5	99	28	SILTY SAND, brown, dry, dense, coarse grained.
			25	3	98	22	moist, loose, some organics
14.0'		6	24	2	97	18	light brown, fine to medium grained, decomposed rock, occasional cobbles to 9"

CBT 87-2

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
	SC-SM		23	4	99	36	SILTY CLAYEY SAND, light brown to brown, damp, medium dense, few cobbles and boulders to 24" from 1.5' to 3.0'
3.0'	SC	7	27	9	100	49	depth. CLAYEY SAND, brown to dark brown, damp, very dense, very cohesive.
6.0			25	5	97	30	SILTY SAND, brown, damp, very dense, well cemented, weathered gravel, decomposed weathered rock
	SM	8	24	3	99	36	dark brown, cohesive, weathered gravel.
11.0'							
12'	SC	7	27	8	99	31	CLAYEY SAND, light brown to brown, damp, dense, less cohesive, medium to
	SC-SM		27	7	98	34	fine grained sand.
14.0'			25	4	98	19	
16.0'	SM		25	2	88	13	SILTY CLAYEY SAND, light brown to brown, damp, medium dense, lightly cemented, slight cohesion. light brown, loose to medium dense, no cementation. SILTY SAND, light brown, damp, loose to medium dense, slight to no cohesion, weathered gravel to 3", few cobbles to 5".

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
			24	2	9		
	SM	8	21	3	9		
			23	3	10		
		9	25	1	9		
11.0'			25	3	9		
	SC-SM	9	24	4	9		
15.0'			25	6	9		

CBT 87-5

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
			22	4	100	44	SILTY CLAYEY SAND, light brown, moist, gravel to 1/4", roots above 3".
	SC-SM		22	6	100	48	brown, low dry strength.
		8	24	7	100	48	loose to medium dense, some cohesion, occasional gravel to 3/8".
8.0'		12	27	6	100	41	loose to dense, gravel to 1 1/2".
11.0'	SM	7	27	3	99	30	SILTY SAND, brown, moist, loose, gravels to 1 1/2", 1-30"x18" boulder.
14.0'	SC-SM		26	7	99	26	SILTY CLAYEY SAND, brown, moist, loose, no cohesion, subrounded gravel to 1/2"
17.0'	SM		25	2	98	20	SILTY SAND, brown, moist, loose, no cohesion, occasional gravel to 3/8"

CBT 87-6

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
	SM	7		NP	92	16	SILTY SAND, brown, moist, loose, cohesive, occasional gravel to 2", roots above 1"
3.0'				NP	98	19	
	SW-SM	3		NP	96	7	WELL GRADED SAND WITH SILT, light brown, gravel to 2"
9.0'				NP	90	5	damp, occasional boulders to 15"
12.0'	SC		28	8	96	20	CLAYEY SAND, reddish brown, very moist, loose, gravel to 2", 5% cobbles
14.0'	SM		22	1	98	22	SILTY SAND, reddish brown, very moist, cohesive, occasional cobbles.
			26	2	100	35	damp, loose, slight cohesion

DEPTH	LOG	MC	LL	PI	
			23	2	9
3.0'	SM	5	20	3	9
	SC	4	21	11	9
7.0'					
8.0'	SW-SM	4	20	2	6
10.0'	ML		35	10	10
11.0'	SC		34	14	9
12.0'	ML	13	33	9	10
13.0'	SC-SM		27	6	9

CBT 87-9

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
1.0'	SM		22	2	98	43	SILTY SAND, brown, damp, loose, roots.
2.0'	SC-SM		20	4	98	48	SILTY CLAYEY SAND, brown, dry, medium
	SC	7	24	9	98	48	dense, cohesive, roots and organics.
7.0'		9	30	10	98	46	CLAYEY SAND, dark brown, dry, dense, cohesive, layer of gravel with cobbles to 6"
	SM		28	3	100	38	no cobbles.
10.0'				NP	98	24	SILTY SAND, light brown, dry, slight cohesion, one 5" cobble.
12.0'	SC-SM	9	22	5	99	35	damp, decomposed gravel.
				NP	98	28	SILTY CLAYEY SAND, light brown, damp,
16.0'	SM	9	22	3	99	40	some roots, one 8" weathered cobble. SILTY SAND, light brown, damp, some roots, some gravel to 1/4". dark brown to light brown, dry, dense,
18.0'	SC		23	9	99	35	cohesive. CLAYEY SAND, dark brown, dry, dense, cohesive, some mica particles.

CBT 87-10

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
1.5'	SM		20	2	98	43	SILTY SAND, dark brown, moist, slightly cohesive, low dry strength, some roots, occasional gravels to 3/4" in top 2" depth.
3.0'	CL	7	28	10	100	51	SANDY LEAN CLAY, dark brown, moist, very stiff, highly cohesive, high dry strength.
6.0'	SC		28	10	100	38	CLAYEY SAND, brown, moist, loose to medium dense, some cohesion, low dry strength.
9.0'	SC-SM	7	24	7	97	31	SILTY, CLAYEY SAND, light brown, moist, some cohesion, some subangular gravel to 3/4".
11.5'	SM			NP	98	22	SILTY SAND, light brown, moist, some cohesion, subangular gravel to 1 1/2"
13.5'	SC-SM	11	22	5	100	45	SILTY, CLAYEY SAND, brown, moist, low to medium dry strength
16.0'	SC		29	9	99	32	CLAYEY SAND, brown, moist low to medium dry strength.

DEPTH	LOG	MC	LL	PI
2.0'	SM		21	2
5.0'	SC	9	35	17
8.0'	SC-SM		21	4
12.5'	SM			NP

LUE ENGINEERING PAYS

CBT 87-3

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
	SM		24	2	99	32	SILTY SAND; light brown, damp, loose, no cohesion, fine grass roots throughout.
		8	21	3	99	37	brown, medium dense, slight to moderate cohesion, no roots.
			23	3	100	37	brown to dark brown, medium dense to very dense, cohesive and/or cemented, gravel to 2".
		9	25	1	99	24	brown, no gravel.
11.0'			25	3	99	23	medium dense, slightly cohesive.
	SC-SM	9	24	4	99	20	SILTY CLAYEY SAND; light brown, damp, loose to medium dense, slightly cohesive.
15.0'			25	6	99	32	brown, medium dense, cohesive.

CBT 87-4

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
1.0'	SM		29	5	98	21	SILTY SAND; light brown, damp, loose medium dense, slight cohesion, grass roots throughout.
2.0'	SC		25	8	99	46	
3.0'	SC-SM	12	29	7	100	45	
7.0'	SM		20	NP	100	44	CLAYEY SAND; brown, damp to moist, medium dense, cohesive, gravel to 2". occasional cobble to 5".
9.0'	SC-SM	8	28	7	100	30	SILTY CLAYEY SAND; brown to dark brown, moist, medium dense to dense, cohesive, some weathered gravel to 2".
10.5'	SM		24	3	100	19	SILTY SAND; dark brown to brown, moist to damp, dense, cohesive, some weathered gravel to 2", occasional cobble to 5".
13.0'	SC-SM	9	26	5	99	37	SILTY CLAYEY SAND; brown, moist to damp, cohesive, some weathered, gravel to 2", occasional cobble to 5".
16.0'	SM		24	3	97	27	SILTY SAND; brown, damp, loose to medium dense, slight to no cohesion.
17.0'	SC-SM		25	5	99	25	SILTY CLAYEY SAND, brown, damp to moist, medium dense, cohesive.
18.0'	CL-ML	10	27	6	100	52	SILTY SAND, brown, damp, loose to medium dense, cohesive.
19.0'	SC-SM		25	4	99	32	SILTY CLAYEY SAND, brown damp, loose to medium dense, slight cohesion, coorser sand, some gravel to 2".
							SANDY SILTY CLAY, brown, damp to moist, stiff, cohesive.
							SILTY CLAYEY SAND; brown, damp, medium dense, less cohesive then sample above.

CBT 87-7

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
	SM		23	2	98	25	SILTY SAND; brown, damp, loose, grass roots throughout.
3.0'		5	20	3	99	30	moist, loose to medium dense, cohesive, occasional gravel.
	SC	4	21	11	99	36	CLAYEY SAND; brown, dry, medium dense to dense, cohesive, some gravel.
7.0'							
8.0'	SW-SM	4	20	2	68	11	WELL GRADED SAND WITH SILT AND GRAVEL, light brown, gravel, gravel to 3", 15% cobbles, occasional boulders to 18"
10.0'	ML		36	10	100	56	
11.0'	SC		34	14	99	45	SANDY SILT, brown, damp, stiff, cohesive.
12.0'	ML	13	33	9	100	50	CLAYEY SAND, light brown, damp, cohesive, occasional gravel to 3".
13.0'	SC-SM		27	6	97	25	SANDY SILT, reddish brown, damp, stiff to hard, cohesive.
							SILTY, CLAYEY SAND, reddish brown, damp, loose, trenching stopped due to large boulders.

CBT 87-8

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
2.0'	ML		21	1	99	50	SANDY SILT, light brown, loose, roots.
	CL	7	29	9	100	52	SANDY LEAN CLAY, dark brown, dry, hard, cohesive.
8.0'		8	31	11	100	56	occasional cobble.
	SC		27	9	99	49	CLAYEY SAND, brown, dry, dense, cohesive.
13.0'			27	8	100	40	light brown, damp, medium dense, slight cohesion, some gravel.
15.0'	SM		31	8	100	36	SILTY SAND, light brown, damp, medium dense, slight cohesion, some gravel.

CBT 87-11

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
2.0'	SM		21	2	99	42	SILTY SAND, brown, damp, loose, roots and organics, some gravel.
5.0'	SC	9	35	17	97	45	CLAYEY SAND; dark brown, dry, dense, roots, layer of cobbles at 4.5' depth.
8.0'	SC-SM		21	4	99	35	SILTY, CLAYEY SAND; dark brown, dry, dense, roots, 1-boulder at 8' depth.
12.5'	SM			NP	79	15	SILTY SAND WITH GRAVEL, dry, dense, cobbles and boulders from 9' to 10' depth, drilling stopped due to large boulders.

CBT 87-12

DEPTH	LOG	MC	LL	PI	-4	-200	DESCRIPTION
	SC		23	8	97	47	CLAYEY SAND; brown, damp, slight cohesion, roots throughout sample.
		7	33	14	99	37	brown to dark brown, dry, cohesive, some roots, occasional cobbles, 18" boulder at 4' depth.
7.0'			25	8	99	43	some grave
9.0'	SC-SM	8	24	6	100	46	SILTY, CLAYEY SAND; brown to dark brown, dry, cohesive, some gravel, occasional cobbles.

SCALE 1 IN. = 5 FT.

5 0 5 10 15

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:

DRAWN BY: GJBA

CHECKED BY:

SANTA ANA RIVER, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

IMPERVIOUS BORROW AREA

SOIL LOGS

CBT 87-1 THRU CBT 87-12

SUBMITTED BY: DATE APPROVED:

SPEC. NO. DACW09-____-B-____

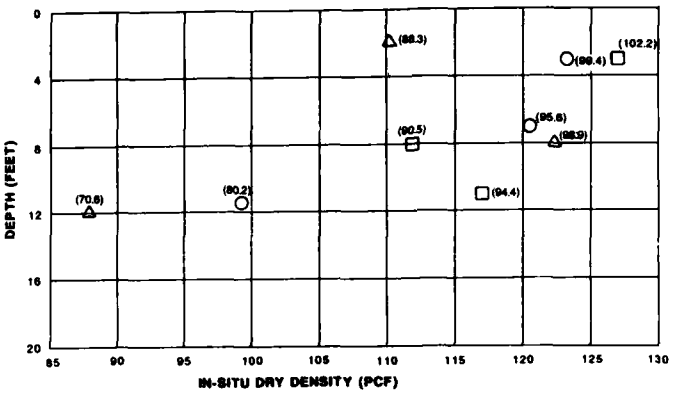
DISTRICT FILE NO.

CHIEF, ____ DIVISION

SHEET

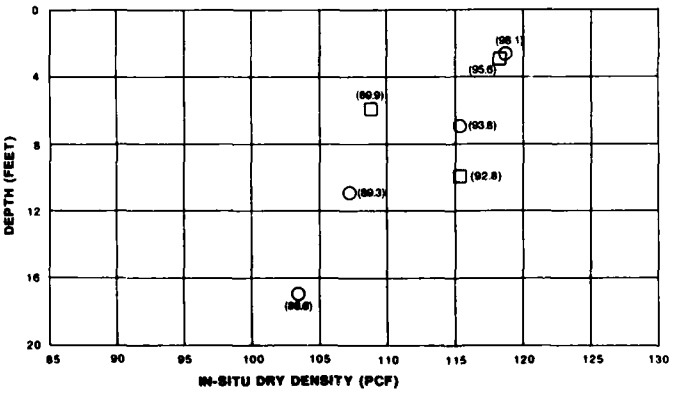
SAFETY PAYS

PLATE A-73



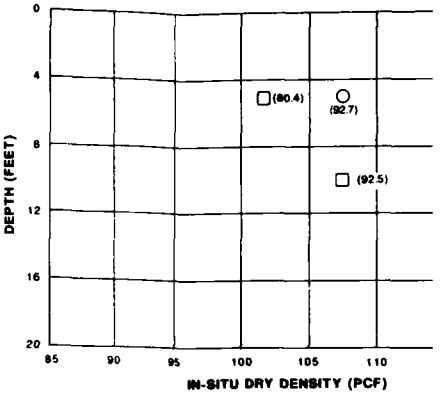
SYMBOL	HOLE/ SAMPLE NUMBER	ELEV./ DEPTH (FT)	IN-SITU DRY DEN. (PCF)	OPT. MOIST. CONT. (%)	MAX. DRY DEN. (PCF)	USCS CLASS.	MECHANICAL ANALYSIS GR	SA	FI	LL	PI	REMARKS
○	CBT87-1	3.0	123.2	12.0	123.9	SC-SM		57	43	23	6	ASTM D 698-78(A)
○	CBT87-1	7.0	120.4	11.0	126.0	SM		72	28	28	5	ASTM D 698-78(A)
○	CBT87-1	11.5	99.3	11.5	123.8	SM	3	79	18	24	2	ASTM D 698-78(A)
□	CBT87-2	3.0	126.8	11.0	124.1	SC		51	49	27	9	ASTM D 698-78(A)
□	CBT87-2	8.0	111.7	11.5	123.5	SM		64	36	24	3	ASTM D 698-78(A)
□	CBT87-2	11.0	116.9	11.2	123.8	SC		69	31	27	8	ASTM D 698-78(A)
△	CBT87-3	2.0	110.2	10.7	124.8	SM		63	37	21	3	ASTM D 698-78(A)
△	CBT87-3	8.0	122.4	12.5	123.8	SM		76	24	25	1	ASTM D 698-78(A)
△	CBT87-3	12.0	97.8	11.8	124.3	SC-SM		68	32	25	6	ASTM D 698-78(A)

CBT87-1 TO 87-3



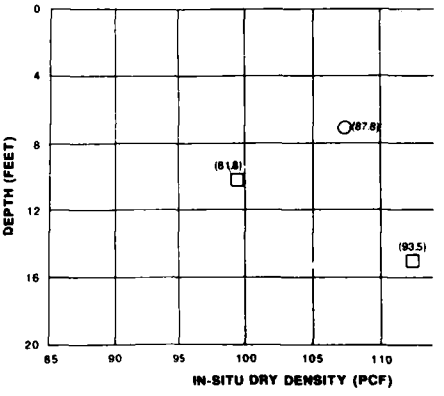
SYMBOL	HOLE/ SAMPLE NUMBER	ELEV./ DEPTH (FT)	IN-SITU DRY DEN. (PCF)	OPT. MOIST. CONT. (%)	MAX. DRY DEN. (PCF)	USCS CLASS.	MECHANICAL ANALYSIS GR	SA	FI	LL	PI	REMARKS
○	CBT87-4	2.5	118.7	12.0	121.0	SC-SM		55	45	29	7	ASTM D 698-78(A)
○	CBT87-4	7.0	115.4	11.5	123.0	SC-SM		70	30	28	7	ASTM D 698-78(A)
○	CBT87-4	11.5	107.3	12.5	120.2	SC-SM	1	62	37	28	5	ASTM D 698-78(A)
○	CBT87-4	17.0	103.6	14.0	117.0	CL-ML		48	52	27	6	ASTM D 698-78(A)
□	CBT87-5	3.0	118.3	12.6	123.8	SC-SM		52	48	24	7	ASTM D 698-78(A)
□	CBT87-5	6.0	109.8	13.5	121.0	SC-SM		59	41	27	6	ASTM D 698-78(A)
□	CBT87-5	9.0	115.3	11.0	124.1	SM	1	69	30	27	3	ASTM D 698-78(A)

CBT87-4 TO 87-5



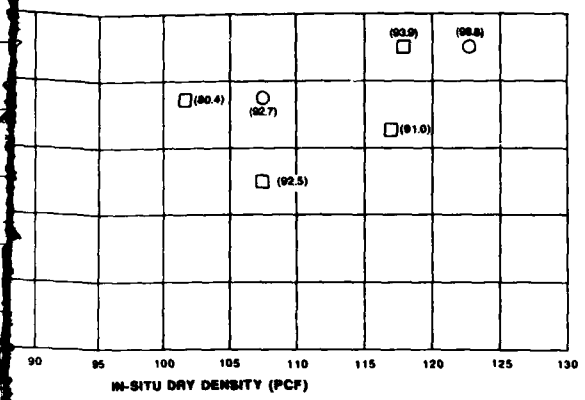
SYMBOL	HOLE/ SAMPLE NUMBER	ELEV./ DEPTH (FT)	IN-SITU DRY DEN. (PCF)	OPT MOIST CONT (%)	MAX. DRY DEN. (PCF)	USCS CLASS.	MECHANICAL ANALYSIS GR	SA
○	CBT87-6	2.0	122.6	9.8	124.1	SM		81
○	CBT87-6	5.0	107.5	9.5	116.0	SW-SM	2	91
□	CBT87-7	2.0	117.7	10.0	125.4	SM	1	69
□	CBT87-7	5.0	101.7	10.5	126.5	SC	1	63
□	CBT87-7	7.0	116.8	9.4	126.4	SW-SM	27	62
□	CBT87-7	10.0	107.3	13.4	116.0	SC	1	54

CBT87-6 TO 87-7



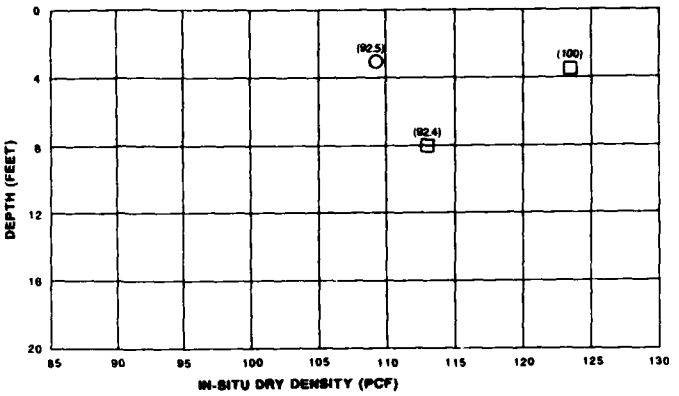
SYMBOL	HOLE/ SAMPLE NUMBER	ELEV./ DEPTH (FT)	IN-SITU DRY DEN. (PCF)	OPT. MOIST. CONT. (%)	MAX. DRY DEN. (PCF)	USCS CLASS.	MECHANICAL ANALYSIS GR	SA
○	CBT87-8	3.0	125.7	11.5	122.4	CL		48
○	CBT87-8	7.0	107.4	12.5	122.4	CL		44
□	CBT87-9	2.0	123.2	11.7	124.0	SC		51
□	CBT87-9	6.0	116.9	12.8	120.0	SC	1	53
□	CBT87-9	10.0	99.4	11.5	121.5	SC-SM	1	64
□	CBT87-9	15.0	112.2	12.5	120.0	SM		60
△	CBT87-10	2.0	116.7	13.0	120.5	SM	2	55
△	CBT87-10	6.0	117.4	12.3	122.7	SC-SM	2	67
△	CBT87-10	12.0	119.6	12.2	121.5	SC-SM		55

CBT87-8 TO 87-10



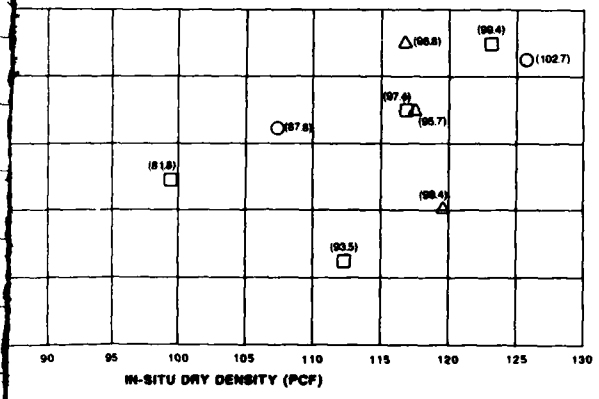
HOLE/ SAMPLE NUMBER	ELEV./ DEPTH (FT)	IN-SITU DRY DEN. (PCF)	OPT. MOIST. CONT. (%)	MAX. DRY DEN. (PCF)	USCS CLASS.	MECHANICAL ANALYSIS GR	SA	FI	LL	PI	REMARKS
CBT87-6	2.0	122.6	9.8	124.1	SM		81	19			ASTM D 698-78(A)
CBT87-6	5.0	107.5	9.5	116.0	SW-SM	2	91	7			ASTM D 698-78(A)
CBT87-7	2.0	117.7	10.0	125.4	SM	1	69	30	20	3	ASTM D 698-78(A)
CBT87-7	5.0	101.7	10.5	126.5	SC	1	63	36	21	11	ASTM D 698-78(A)
CBT87-7	7.0	116.8	9.4	128.4	SW-SM	27	62	11	20	2	ASTM D 698-78(A)
CBT87-7	10.0	107.3	13.4	116.0	SC	1	54	45	34	14	ASTM D 698-78(A)

CBT87-6 TO 87-7



SYMBOL	HOLE/ SAMPLE NUMBER	ELEV./ DEPTH (FT)	IN-SITU DRY DEN. (PCF)	OPT. MOIST. CONT. (%)	MAX. DRY DEN. (PCF)	USCS CLASS.	MECHANICAL ANALYSIS GR	SA	FI	LL	PI	REMARKS
○	CBT87-11	3.0	108.3	13.6	118.2	SC	2	53	45	35	17	ASTM D 698-78(A)
□	CBT87-12	3.5	123.4	11.5	123.3	SC		63	37	33	14	ASTM D 698-78(A)
□	CBT87-12	8.0	113.0	11.6	122.3	SC-SM		54	46	24	6	ASTM D 698-78(A)

CBT87-11 TO 87-12



HOLE/ SAMPLE NUMBER	ELEV./ DEPTH (FT)	IN-SITU DRY DEN. (PCF)	OPT. MOIST. CONT. (%)	MAX. DRY DEN. (PCF)	USCS CLASS.	MECHANICAL ANALYSIS GR	SA	FI	LL	PI	REMARKS
CBT87-8	3.0	125.7	11.5	122.4	CL		48	52	29	9	ASTM D 698-78(A)
CBT87-8	7.0	107.4	12.5	122.4	CL		44	56	31	11	ASTM D 698-78(A)
CBT87-9	2.0	123.2	11.7	124.0	SC		51	49	24	9	ASTM D 698-78(A)
CBT87-9	5.0	116.9	12.8	120.0	SC	1	53	46	30	16	ASTM D 698-78(A)
CBT87-9	10.0	99.4	11.5	121.5	SC-SM	1	64	35	22	5	ASTM D 698-78(A)
CBT87-9	15.0	112.2	12.5	120.0	SM		60	40	22	3	ASTM D 698-78(A)
CBT87-10	2.0	116.7	13.0	120.5	SM	2	55	43	26	2	ASTM D 698-78(A)
CBT87-10	5.0	117.4	12.3	122.7	SC-SM	2	67	31	24	7	ASTM D 698-78(A)
CBT87-10	12.0	119.8	12.2	121.5	SC-SM		55	45	22	5	ASTM D 698-78(A)

CBT87-8 TO 87-10

NOTES:

1. REFER TO PLATE A-60 FOR LOCATION OF TEST TRENCHES
2. REFER TO PLATE A-73 FOR SOIL LOGS OF CBT87-1 TO CBT87-12
3. THE NUMBERS IN PARENTHESES INDICATE THE RELATIVE COMPACTION OR PERCENT COMPACTION AS REPRESENTED BY THE RATIO OF IN-SITU DENSITY AND LABORATORY MAXIMUM DENSITY

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM
IMPERVIOUS BORROW AREA
COMPARISON OF IN-SITU
VS
LABORATORY MAXIMUM DENSITIES

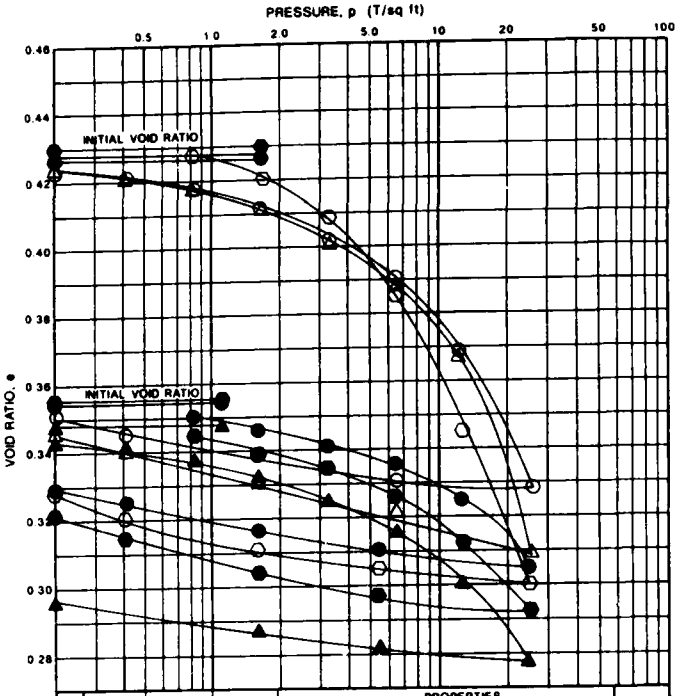
SUBMITTED BY: DATE APPROVED:

SPEC. NO. DACW09-____-____

DISTRICT FILE NO.

SHEET

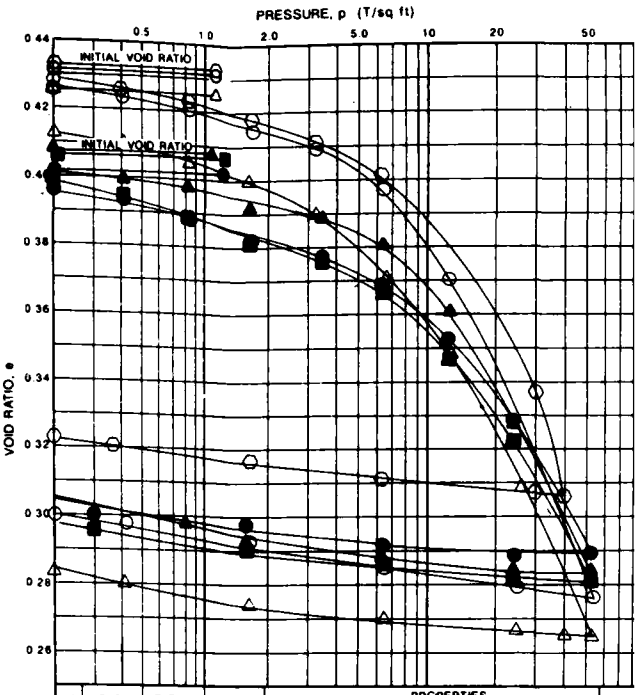
PLATE A-74



SYMBOL	HOLE/ SAMPLE NUMBER	ELEV./ DEPTH (FT)	USCS CLASS.	PROPERTIES BEFORE TEST: DRY DENSITY (PCF)	BEFORE TEST: MOIST CONT (%)	BEFORE TEST: SATURATION (%)	BEFORE TEST: VOID RATIO	AFTER TEST: MOIST CONT (%)	AFTER TEST: VOID RATIO	AFTER TEST: SATURATION (%)	REMARKS
○	TH85-4 & TH85-9	BLEND A	CLAYEY SAND(SC)	117.4	8.1	38.2	0.43	13.5	0.397	88.7	95% MAX -3% OPT
●	TH85-4 & TH85-9	BLEND A	CLAYEY SAND(SC)	124.0	5.8	48.0	0.353	13.8	0.389	95.0	95% MAX -3% OPT
○	TH85-4 & TH85-9	BLEND A	CLAYEY SAND(SC)	117.7	8.9	96.0	0.427	12.9	0.389	95.0	95% MAX @ OPT
●	TH85-4 & TH85-9	BLEND A	CLAYEY SAND(SC)	124.1	8.9	98.0	0.352	12.6	—	95.5	95% MAX @ OPT
△	TH85-4 & TH85-9	BLEND A	CLAYEY SAND(SC)	117.7	12.0	75.0	0.427	13.2	0.388	95.0	95% MAX +3% OPT
▲	TH85-4 & TH85-9	BLEND A	CLAYEY SAND(SC)	124.5	11.3	97.5	0.348	11.8	0.317	100	95% MAX +2.5% OPT

NOTES: MAXIMUM DENSITY = 130.6 PCF (ASTM D 1557-78(A))
OPTIMUM MOISTURE CONTENT = 9.0%

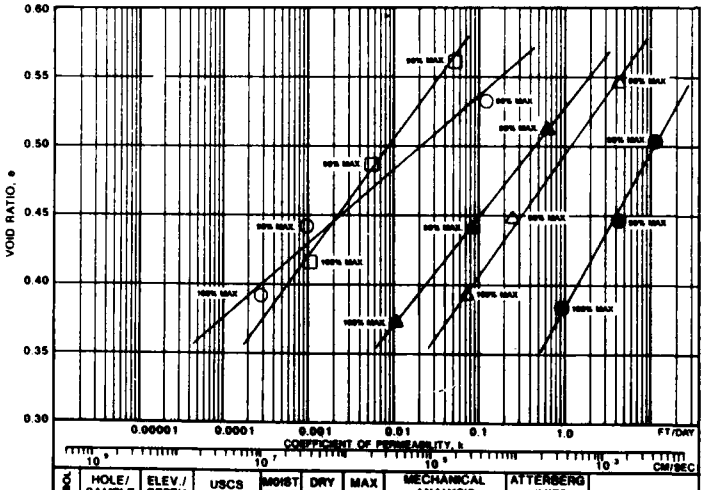
CONSOLIDATION TEST DATA



SYMBOL	HOLE/ SAMPLE NUMBER	ELEV./ DEPTH (FT)	USCS CLASS.	PROPERTIES BEFORE TEST: DRY DENSITY (PCF)	BEFORE TEST: MOIST CONT (%)	BEFORE TEST: SATURATION (%)	BEFORE TEST: VOID RATIO	AFTER TEST: MOIST CONT (%)	AFTER TEST: VOID RATIO	AFTER TEST: SATURATION (%)	REMARKS
○	CBT86-30 100030	5.0'-11.0'	CLAYEY SAND(SC)	116.7	10.6	85.0	0.433	11.9	0.339	94.0	95% M -17% O
○	CBT86-30 100030	5.0'-11.0'	CLAYEY SAND(SC)	116.7	12.5	77.6	0.432	11.7	0.316	99.2	95% M @ OP
△	CBT86-30 100030	5.0'-11.0'	CLAYEY SAND(SC)	116.9	14.3	90.1	0.425	10.8	0.288	100	95% M +2% O
●	CBH86-38	BLEND 8A	SILTY SAND (SM)	118.8	10.9	73	0.401	11.0	0.306	96	95%M @ OP
■	CBH86-38	BLEND 8A	SILTY SAND (SM)	118.9	12.9	85	0.407	11.4	0.302	100	95%M OPT.
▲	CBH86-38	BLEND 8A	SILTY SAND (SM)	119.8	9.0	59.0	0.408	10.6	0.304	94.0	95%M OPT.

NOTES: CBT86-30 MAXIMUM DENSITY = 118.9 PCF ASTM D 698-78(A)
OPTIMUM MOISTURE CONTENT = 12.7%
CBH86-38 MAXIMUM DENSITY = 122.0 PCF ASTM D 698-78(A)
OPTIMUM MOISTURE CONTENT = 11.0%

CONSOLIDATION TEST DATA

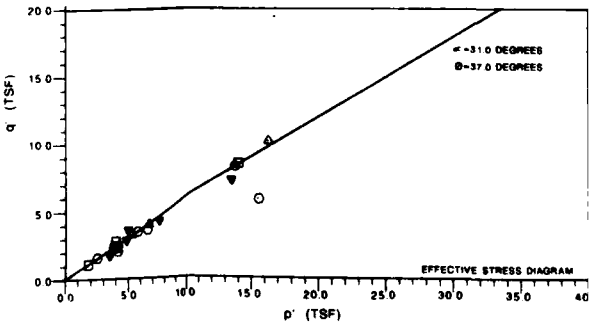


SYMBOL	HOLE/ SAMPLE NUMBER	ELEV./ DEPTH (FT)	USCS CLASS	MOIST CONT (%)	DRY DENS (PCF)	MAX PART (IN.)	MECHANICAL ANALYSIS: GR	SA	FI	ATTERBERG LIMITS: LL	PI	REMARKS
△	TH-86-9 96586/87	COMP BLEND	SILTY SAND (SM)	12.2	120.8	1.0	4	74	22		NP	
○	CBT86-34 100045/46	0.75'-5'	SANDY CLAY (CL)	8.8	120.8	0.187		46	52	27	11	
□	CBT86-30 100030	BLEND 2A	CLAYEY SAND (SC)	12.7	118.9	2.0	3	73	24	33	13	
▲	CBH86-29 100035	BLEND 7A	CLAYEY SAND (SC-SM)	11.0	122.1	1.0	5	56	39	22	6	
●	CBH86-38 100050	BLEND 8A	SILTY SAND (SM)	11.0	122.0	6.0	5	77	18		NP	

PERMEABILITY TEST DATA

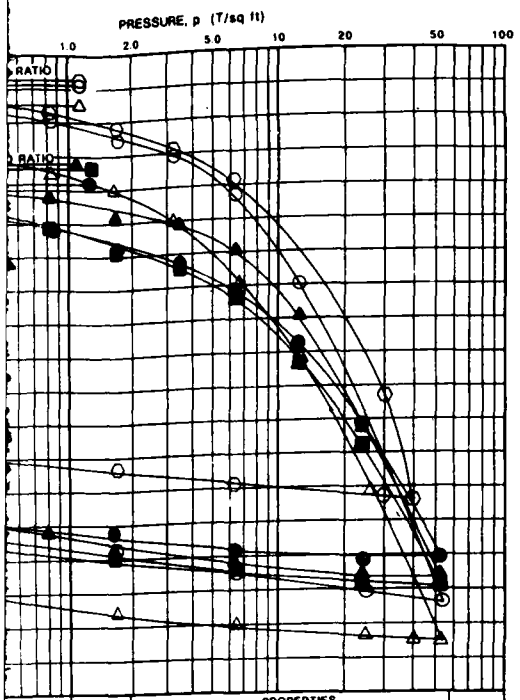
LEGEND FOR P-Q DIAGRAMS

▼ CBT86-34 100045/46	100% MAX. @ OPT	○ CBT86-30 100030	98% MAX. @ OPT
□ CBH86-29 100035	98% MAX. @ OPT. -2%	△ CBT86-36 100050	98% MAX @ OPT
⬡ TH85-4/9 96586/87	98% MAX @ OPT. +1.5%		



p'-q' DIAGRAM

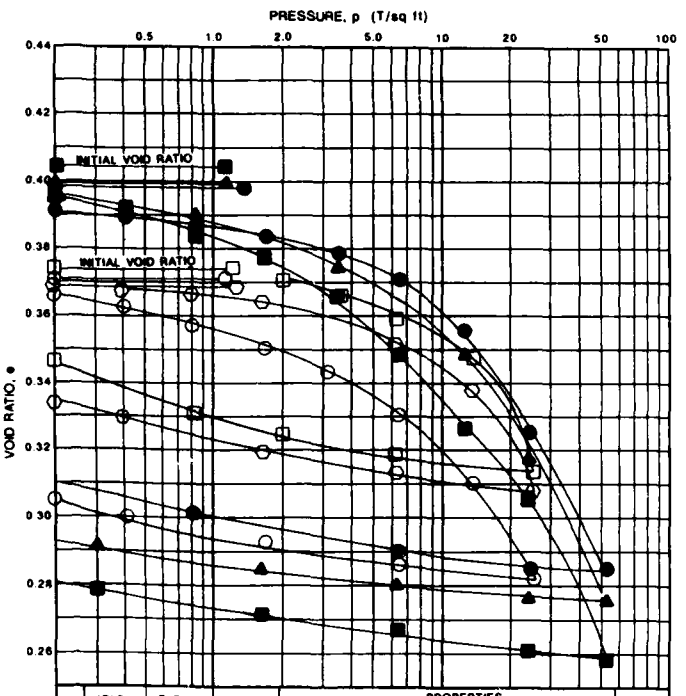
ENHANCEMENT THRU ENGINEERING



ELEV / DEPTH (FT)	USCS CLASS.	PROPERTIES BEFORE TEST: DRY DENSITY (PCF)	MOIST CONT (%)	SATUR-ATION (%)	VOID RATIO	AFTER TEST: MOIST CONT (%)	VOID RATIO	SATUR-ATION (%)	REMARKS
5/-110'	CLAYEY SAND(SC)	116.7	10.6	85.0	0.433	11.9	0.399	94.0	98% MAX -1.7% OPT
5.0'-110'	CLAYEY SAND(SC)	116.7	12.5	77.6	0.432	11.7	0.315	98.2	98% MAX @ OPT
5.0'-110'	CLAYEY SAND(SC)	116.9	14.3	90.1	0.425	10.8	0.288	100	98% MAX +2% OPT
BLEND 6A	SILTY SAND (SM)	118.8	10.9	73	0.401	11.0	0.308	96	98%MAX @ OPT
BLEND 6A	SILTY SAND (SM)	118.9	12.9	95	0.407	11.4	0.302	100	98%MAX OPT +2%
BLEND 6A	SILTY SAND (SM)	118.8	9.0	59.0	0.406	10.6	0.304	94.0	98%MAX OPT- 2%

30 MAXIMUM DENSITY=118.9 PCF ASTM D 698-78(A)
OPTIMUM MOISTURE CONTENT=12.7%
38 MAXIMUM DENSITY=122.0 PCF ASTM D 698-78(A)
OPTIMUM MOISTURE CONTENT=11.0%

CONSOLIDATION TEST DATA



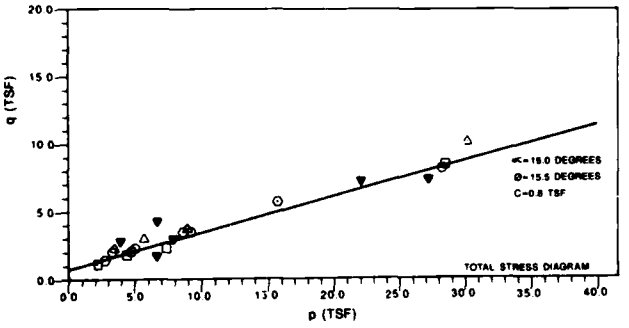
SYMBOL	HOLE/ SAMPLE NUMBER	ELEV / DEPTH (FT)	USCS CLASS.	PROPERTIES BEFORE TEST: DRY DENSITY (PCF)	MOIST CONT (%)	SATUR-ATION (%)	VOID RATIO	AFTER TEST: MOIST CONT (%)	VOID RATIO	SATUR-ATION (%)	REMARKS
□	CBT86-34 100045'46	0.75-5.0	SANDY CLAY(CL)	122.7	9.0	85	0.373	14.1	0.381	100	101% MAX -3.7% OPT
⬡	CBT86-34 100045'46	0.75-5.0'	SANDY CLAY(CL)	123.0	10.7	78	0.369	13.4	0.365	99	101% MAX -2% OPT
○	CBT86-34 100045'46	0.75-5.0	SANDY CLAY(CL)	122.9	12.9	94	0.371	11.6	0.313	100	101% MAX OPT M C
●	CBH86-29	BLEND 7A	CLAYEY SAND (SC-SM)	119.5	11.6	78	0.399	11.0	0.294	100	98%MAX @ OPT
■	CBH86-29	BLEND 7A	CLAYEY SAND (SC-SM)	119.1	14.0	93	0.404	10.6	0.283	100	98%MAX OPT +2%
▲	CBH86-29	BLEND 7A	CLAYEY SAND (SC-SM)	119	9.4	63.9	0.396	11.3	0.327	92.8	98%MAX OPT -2%

NOTES. CBT86-34 MAXIMUM DENSITY=120.6 PCF ASTM D 698-78(A)
OPTIMUM MOISTURE CONTENT=12.2%
CBH86-29 MAXIMUM DENSITY=122.1 PCF ASTM D 698-78(A)
OPTIMUM MOISTURE CONTENT=11.5%

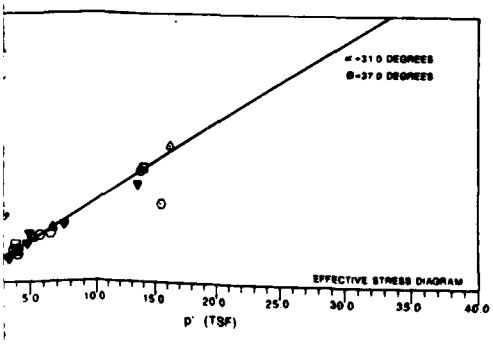
CONSOLIDATION TEST DATA

LEGEND FOR P-Q DIAGRAMS

4 46	100% MAX. @ OPT	○	CBT86-30 100030	98% MAX. @ OPT. -2%
9	98% MAX. @ OPT. -2%	△	CBT86-36 100050	98% MAX. @ OPT. -2%
9 97	98% MAX. @ OPT. +1.5%			



p-q DIAGRAM



p'-q' DIAGRAM

NOTES:

1. REFER TO PLATE A-60 FOR LOCATION OF TEST HOLES.
2. REFER TO PLATE A-62 THRU A-72 FOR SOIL LOGS.
3. REFER TO PLATE A-73 FOR SAMPLE GRADATIONS AND MOISTURE-DENSITY RELATIONSHIPS.

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

IMPERVIOUS BORROW AREA
RESULTS OF DETAILED LABORATORY TESTS

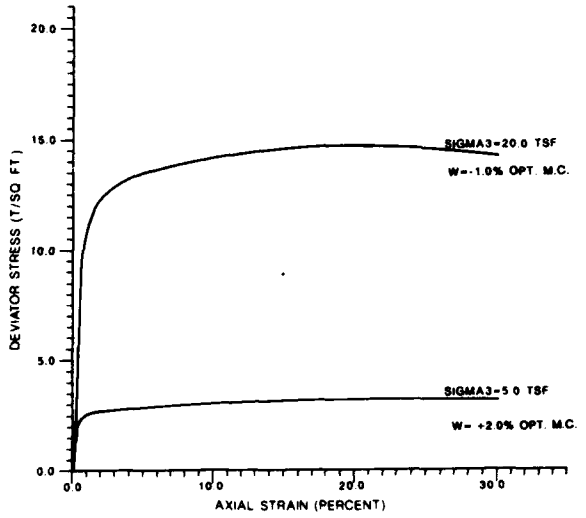
SUBMITTED BY: DATE APPROVED:
CHIEF, SECTION

SPEC. NO. DACW09-___-___-____
DISTRICT FILE NO.

SHEET

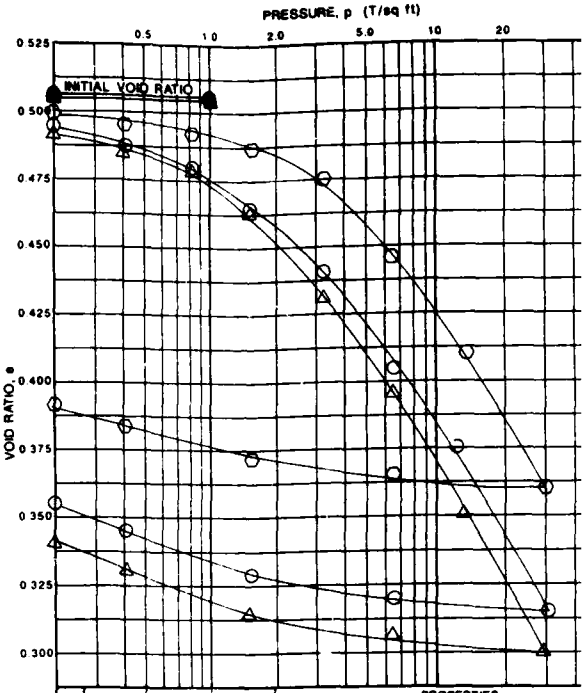
PLATE A-75

ENVIRONMENTAL
ENHANCEMENT
THRU ENGINEERING



CBT86-34 (3% BENTONITE AMENDED)

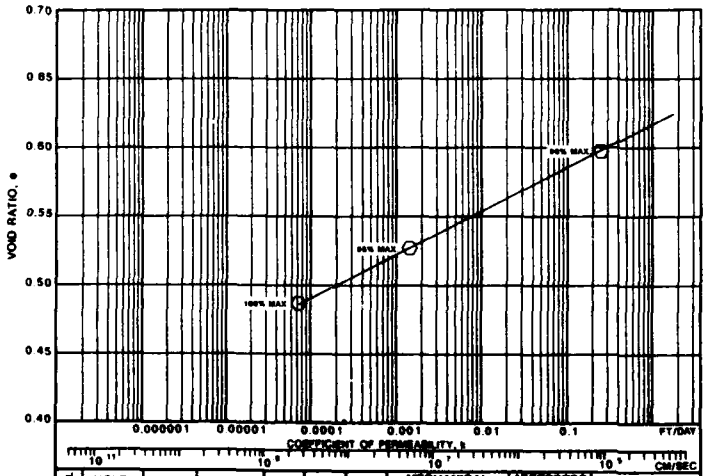
SANDY CLAY (CL) @ 98% MAX. DENSITY



SYMBOL	HOLE/ SAMPLE NUMBER	ELEV DEPTH (FT)	USCS CLASS.	PROPERTIES BEFORE TEST: DRY DENSITY (PCF)	MOIST CONT (%)	SATURATION (%)	VOID RATIO	AFTER TEST: MOIST CONT (%)	VOID RATIO
○	CBT86-34	0.75'-5.0'	SANDY CLAY(CL)	111.1	12.3	66.1	0.505	13.9	0.417
○	CBT86-34	0.75'-5.0'	SANDY CLAY(CL)	111.2	14.3	75.8	0.504	14.5	0.395
△	CBT86-34	0.75'-5.0'	SANDY CLAY(CL)	111.2	16.2	86.2	0.504	13.8	0.389

NOTES BLEND 3B
MAXIMUM DENSITY=113.4 PCF (ASTM D 698-78(A))
OPTIMUM MOISTURE CONTENT=13.7%

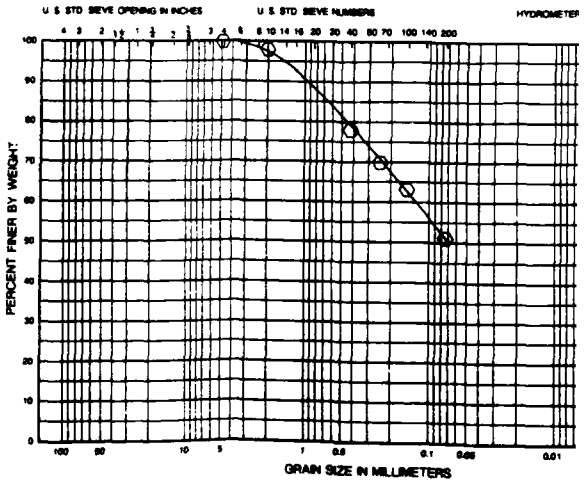
CONSOLIDATION TEST DATA



SYMBOL	HOLE/ SAMPLE NUMBER	ELEV./ DEPTH (FT)	USCS CLASS	MOIST CONT (%)	DRY DENS (PCF)	MAX PART (IN.)	MECHANICAL ANALYSIS: GR	SA	FI	ATTERBERG LIMITS: LL	PI	REMARKS
○	CBT86-34	0.75'-5' BLEND 3B	SANDY CLAY (CL)	13.7	113.4	0.617		49	51	30	13	ASTM D 698-78(A)

NOTES:

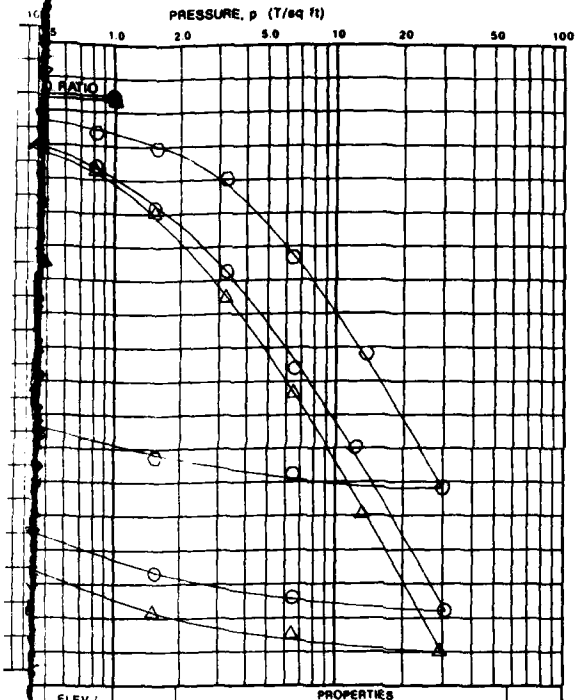
PERMEABILITY TEST DATA



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

GRADATION CURVES

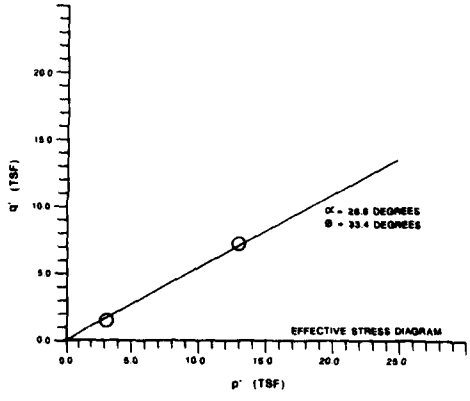
CBT86-34 BLEND 3B



	ELEV / DEPTH (FT)	USCS CLASS.	PROPERTIES BEFORE TEST: DRY DENSITY (PCF)	BEFORE TEST: MOIST CONT (%)	BEFORE TEST: SATUR-ATION (%)	BEFORE TEST: VOID RATIO	AFTER TEST: MOIST CONT (%)	AFTER TEST: VOID RATIO	AFTER TEST: SATUR-ATION (%)	REMARKS
MAX OPT	0.75'-5.0'	SANDY CLAY(CL)	111.1	12.3	65.1	0.506	13.9	0.417	88.5	98% MAX. -2% OPT
MAX OPT	0.75'-5.0'	SANDY CLAY(CL)	111.2	14.3	75.8	0.504	14.5	0.395	98.4	98% MAX @ OPT
MAX OPT	0.75'-5.0'	SANDY CLAY(CL)	111.2	15.2	86.2	0.504	13.8	0.389	100	98% MAX +2% OPT

ND 38
IMUM DENSITY=113.4 PCF (ASTM D 698-78(A))
IMUM MOISTURE CONTENT=13.7%

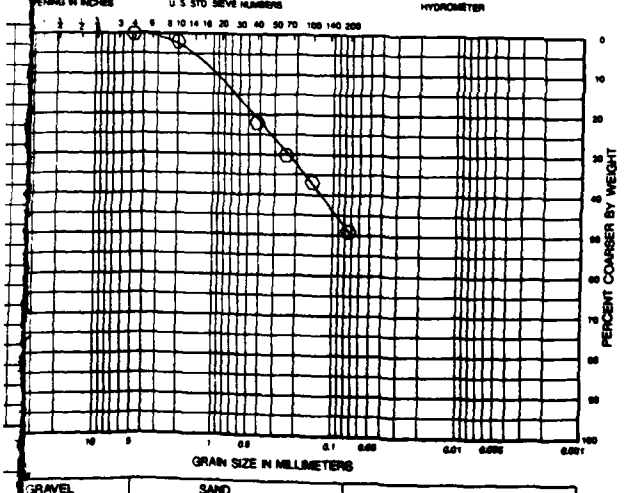
CONSOLIDATION TEST DATA



p' - q' DIAGRAM

CBT86-34 (3% BENTONITE AMENDED)

SANDY CLAY (CL) @ 98% MAX. DENSITY



GRAVEL		SAND			SILT OR CLAY
SE	FINE	COARSE	MEDIUM	FINE	

GRADATION CURVES

CBT86-34 BLEND 38

NOTES:

1. REFER TO PLATE A-60 FOR LOCATION OF TEST HOLES WHERE THE NATURAL SAMPLES WERE OBTAINED.
2. REFER TO PLATE A-62 THRU A-72 FOR SOIL LOGS OF NATURAL SAMPLES.
3. REFER TO PLATE A-79 FOR BENTONITE AMENDED SAMPLE GRADATIONS AND MOISTURE-DENSITY RELATIONSHIPS.

SYMBOL	REVISIONS	DATE	APPROVAL

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

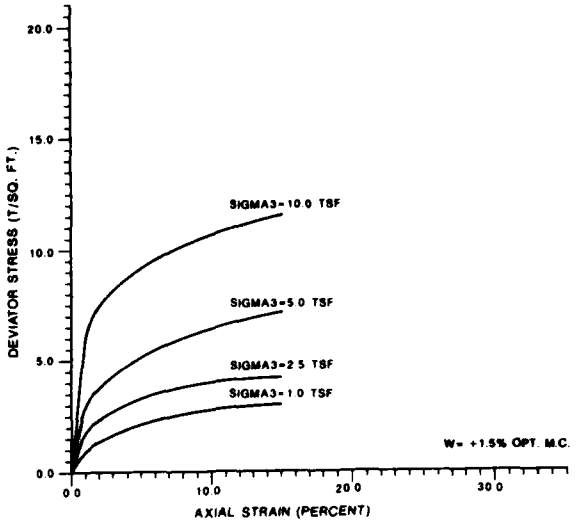
SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM
IMPERVIOUS BORROW AREA
RESULTS OF DETAILED LABORATORY TESTS
(BENTONITE AMENDED)

SUBMITTED BY: DATE APPROVED:
SPEC. NO. SWCW 89- -
DISTRICT FILE NO.
SHEET

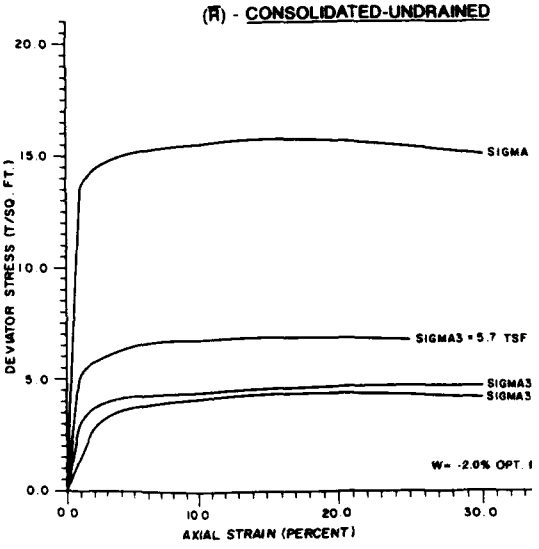
PLATE A-78

($\bar{R}$) - CONSOLIDATED-UNDRAINED



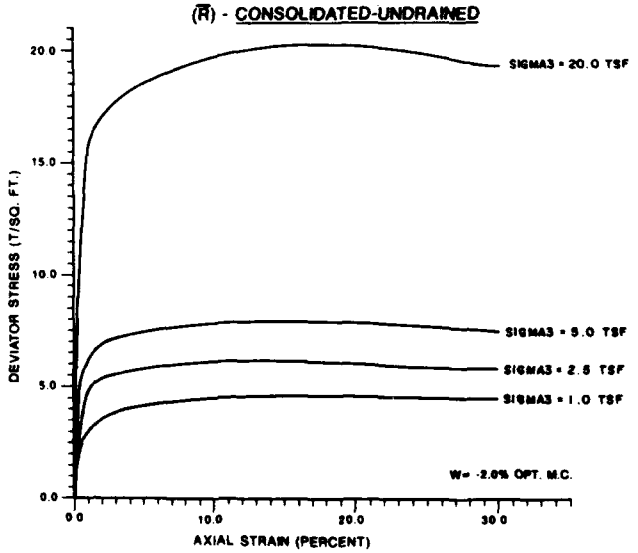
TH85-4 & 9

($\bar{R}$) - CONSOLIDATED-UNDRAINED



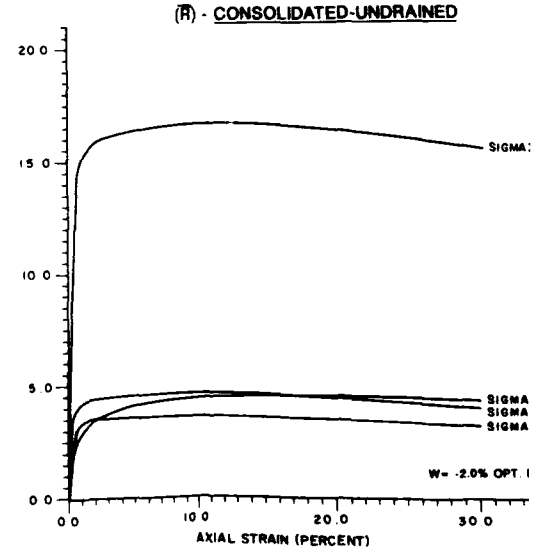
CBT 86-30

($\bar{R}$) - CONSOLIDATED-UNDRAINED



CBT86-38

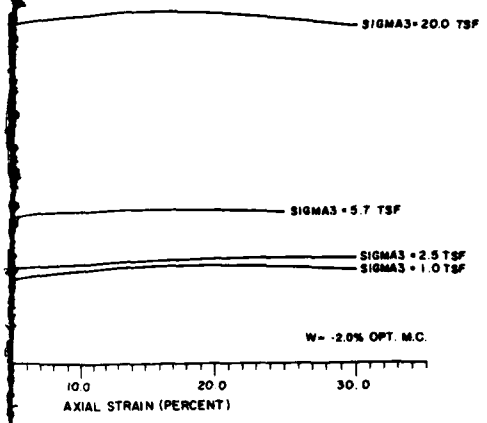
($\bar{R}$) - CONSOLIDATED-UNDRAINED



CBT 86-29

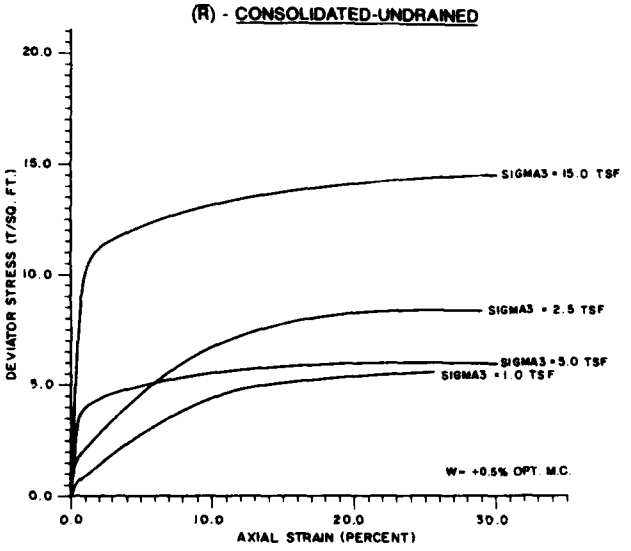
ALUE ENGINEERING PAYS

(R̄) - CONSOLIDATED-UNDRAINED



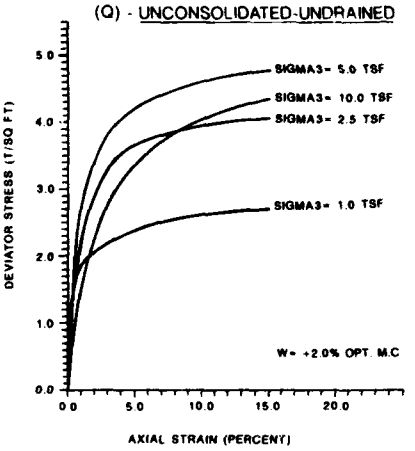
CBT 86-30

(R̄) - CONSOLIDATED-UNDRAINED



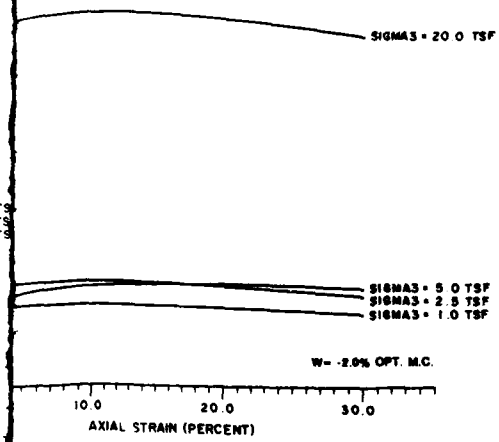
CBT 86-34

(Q) - UNCONSOLIDATED-UNDRAINED



TH 85-21 & 23

(R̄) - CONSOLIDATED-UNDRAINED



CBT 86-29

NOTES:

1. TRIAXIAL COMPRESSION SAMPLES WERE REMOLDED TO 96% OF MAXIMUM DENSITY AS DETERMINED BY ASTM METHOD D 698-78. REMOLDED MOISTURE CONTENTS FOR EACH SAMPLE ARE SHOWN ON THE DRAWINGS.
2. REFER TO PLATE A-78 FOR MOISTURE DENSITY RELATIONSHIPS AND SAMPLE GRADATIONS.
3. REFER TO PLATE A-60 FOR LOCATION OF TEST HOLES.
4. REFER TO PLATES A-62 THRU A-72 FOR SOIL LOGS.

SYMBOL	DESCRIPTION	DATE	APPROVAL
REVISIONS			

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

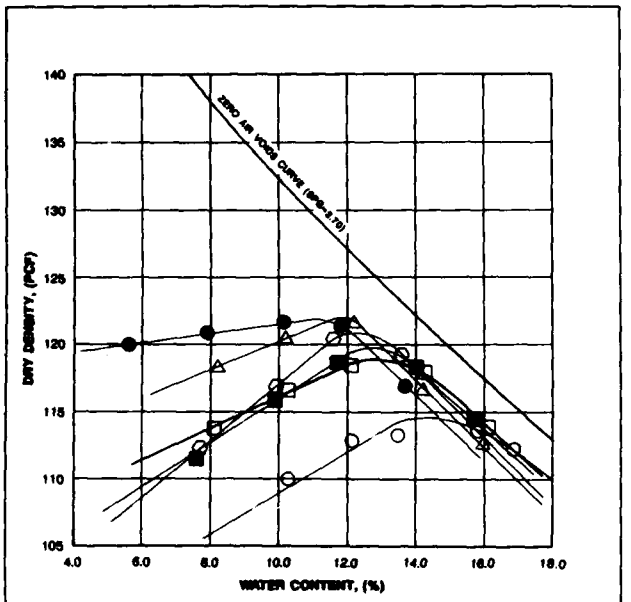
SEVEN OAKS DAM

IMPERVIOUS BORROW AREA
STRESS-STRAIN RELATIONSHIPS

SUBMITTED BY:
DATE APPROVED:
SPEC. NO. DACW09-___-___
DISTRICT FILE NO.
SHEET

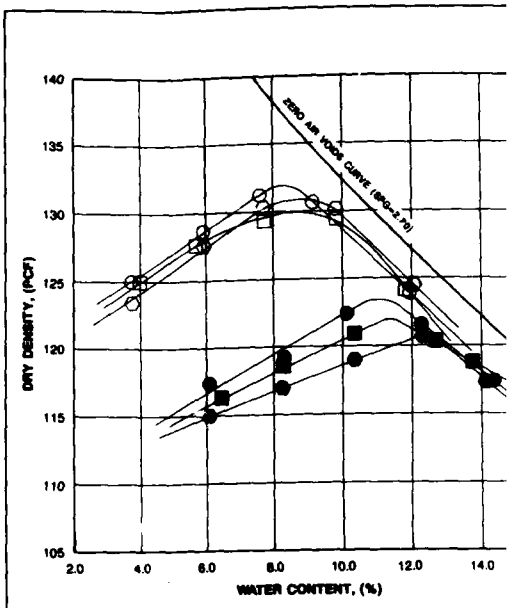
SAFETY PAYS

PLATE A-77



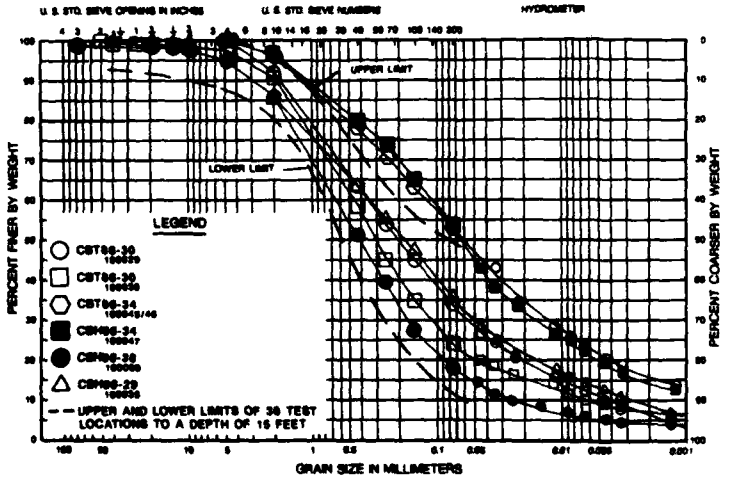
SYMBOL	HOLE/ SAMPLE NUMBER	ELEV./ DEPTH (FT)	OPT. MOIST CONT (%)	MAX. DRY DEN. (PCF)	USCS CLASS.	%> No. 4 SIEVE	%< No.200 SIEVE	LL	PI	Sp G	REMARKS
○	CST86-30 100029	0.75'-5'	14.1	114.7	CLAYEY SAND (SC)	0	34	33	11	2.68	ASTM D 698-78(A)
□	CST86-30 100030	5'-11'	12.7	118.9	CLAYEY SAND (SC)	3	24	33	13	2.68	ASTM D 698-78(A)
⬡	CST86-34 100045/46	0.75'-5'	12.2	120.8	CLY. SAND AND CLAY (SC/CL)	0	52	27	11	2.70	ASTM D 698-78(A)
■	CSH86-34 100047	27.5'-32'	12.9	119.5	SANDY CLAY (CL)	0	53	27	11	2.69	ASTM D 698-78(A)
●	CSH86-36 100059	7'-12'	11.0	122.0	SILTY SAND (SM)	5	18		NP	2.68	ASTM D 698-78(A)
△	CSH86-29 100039	0'-10'	11.8	122.1	CLAYEY SAND (SC-SM)	5	36	22	6	2.73	ASTM D 698-78(A)

COMPACTION TEST DATA



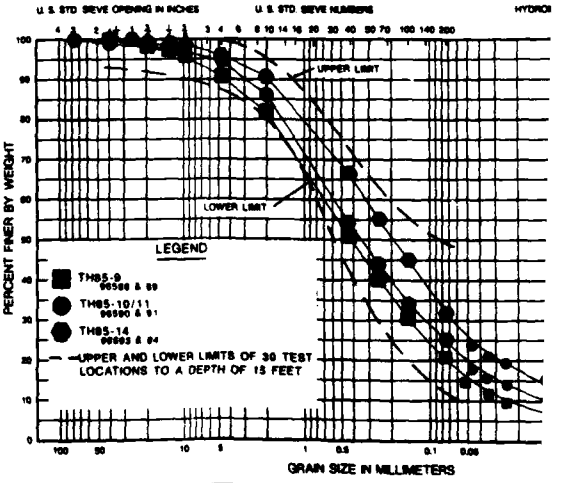
SYMBOL	HOLE/ SAMPLE NUMBER	ELEV./ DEPTH (FT)	OPT MOIST CONT (%)	MAX DRY DEN. (PCF)	USCS CLASS	%> No. 4 SIEVE	%< No.200 SIEVE	LL	PI	Sp G
■	THBS-9 99098 & 99	15'-26'	11.4	121.5	SILTY SAND (SM)	9	22		NP	2.71
□	THBS-9 99099 & 99	15'-26'	9.0	129.8	SILTY SAND (SM)	9	22		NP	2.71
●	THBS-10 THBS-11 99089 & 91	16'-21' 20'	10.9	123.3	SILTY SAND (SM)	5	25		NP	2.69
○	THBS-10 THBS-11 99089 & 91	16'-21' 20'	8.4	131.6	SILTY SAND (SM)	5	25		NP	2.69
⬢	THBS-14 THBS-14 99093 & 94	19' 44'	12.3	120.8	SILTY SAND (SM)	4	32		NP	2.71
⬡	THBS-14 THBS-14 99093 & 94	19' 44'	9.0	130.9	SILTY SAND (SM)	4	32		NP	2.71

COMPACTION TEST DATA



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

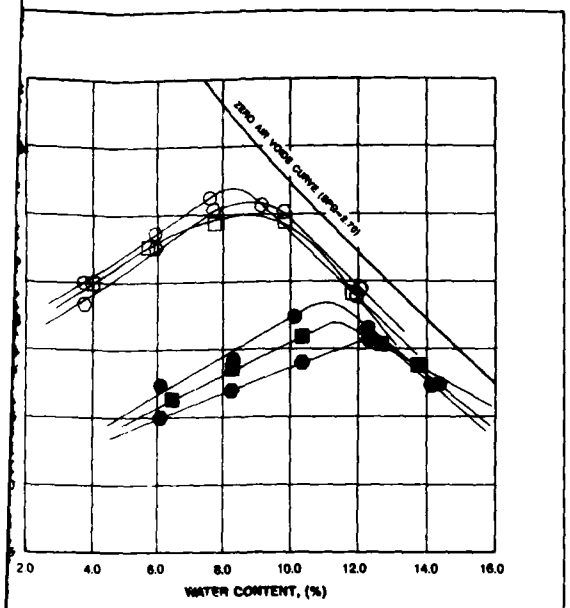
GRADATION CURVES



GRAVEL		SAND			SILT OR
COARSE	FINE	COARSE	MEDIUM	FINE	

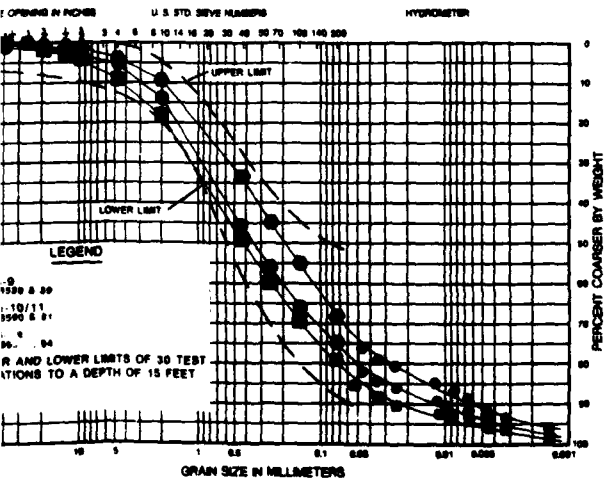
GRADATION CURVES

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

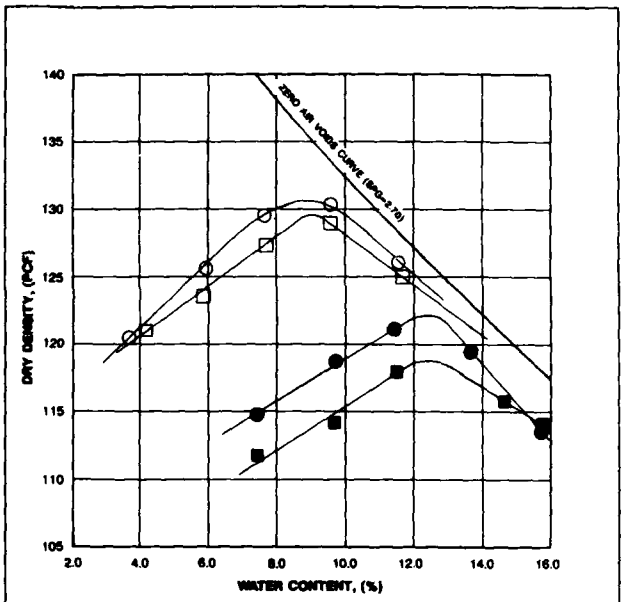


OLE/ MPLE MBER	ELEV./ DEPTH (FT)	OPT MOIST CONT (%)	MAX DRY DEN (PCF)	USCS CLASS.	%> No. 4 SIEVE	%< No.200 SIEVE	LL	PI	Sp. G.	REMARKS
85-9 & 89	15'-26'	11.4	121.5	SILTY SAND (SM)	9	22		NP	2.71	ASTM D 698-78(A)
85-9 & 89	15'-26'	9.0	129.8	SILTY SAND (SM)	9	22		NP	2.71	ASTM D 1557-78(A)
85-10 85-11 & 91	18'-21' 20'	10.9	123.3	SILTY SAND (SM)	5	25		NP	2.69	ASTM D 698-78(A)
85-10 85-11 & 91	18'-21' 20'	8.4	131.6	SILTY SAND (SM)	5	25		NP	2.69	ASTM D 1557-78(A)
85-14 & 94	18' 44'	12.3	120.8	SILTY SAND (SM)	4	32		NP	2.71	ASTM D 698-78(A)
85-14 & 94	18' 44'	9.0	130.9	SILTY SAND (SM)	4	32		NP	2.71	ASTM D 1557-78(A)

COMPACTION TEST DATA

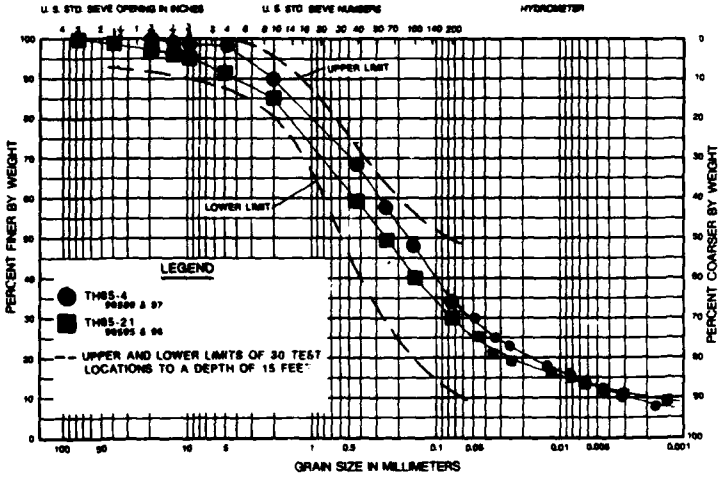


GRADATION CURVES



SYMBOL	HOLE/ SAMPLE NUMBER	ELEV./ DEPTH (FT)	OPT MOIST CONT (%)	MAX DRY DEN (PCF)	USCS CLASS	%> No 4 SIEVE	%< No.200 SIEVE	LL	PI	Sp G	REMARKS
●	TH85-4 88&89 & 97	BLEND A	12.5	121.6	CLAYEY SAND (SC)	2	34	27	9	2.69	ASTM D 698-78(A)
○	TH85-4 88&89 & 97	BLEND A	9.0	130.6	CLAYEY SAND (SC)	2	34	27	9	2.69	ASTM D 1557-78(A)
■	TH85-21 88&89 & 96 TH85-23 88&89 & 96	0'-13' 0'-15'	12.3	119.0	CLAYEY SAND (SC)	8	30	32	14	2.71	ASTM D 698-78(A)
□	TH85-21 88&89 & 96 TH85-23 88&89 & 96	0'-13' 0'-15'	9.0	129.6	CLAYEY SAND (SC)	8	30	32	14	2.71	ASTM D 1557-78(A)

COMPACTION TEST DATA



GRADATION CURVES

NOTES:

1. REFER TO PLATE A-80 FOR LOCATION OF TEST HOLES.
2. REFER TO PLATES A-62 THRU A-72 FOR SOIL LOGS.

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

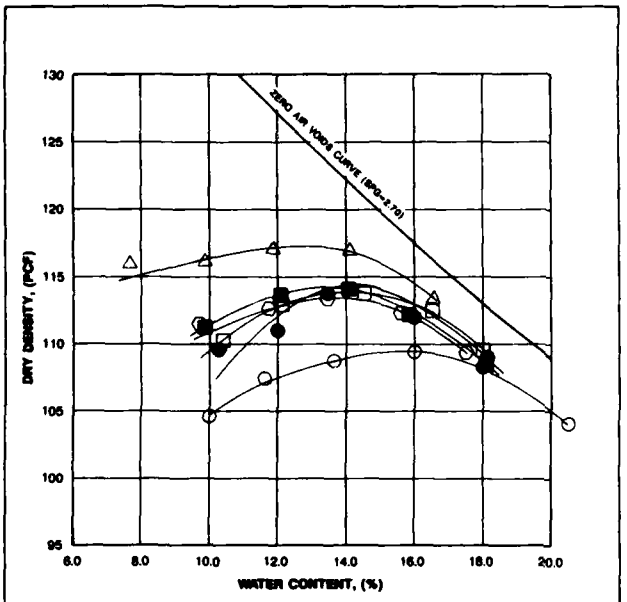
SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE I, GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

IMPERVIOUS BORROW AREA
MOISTURE-DENSITY RELATIONSHIPS

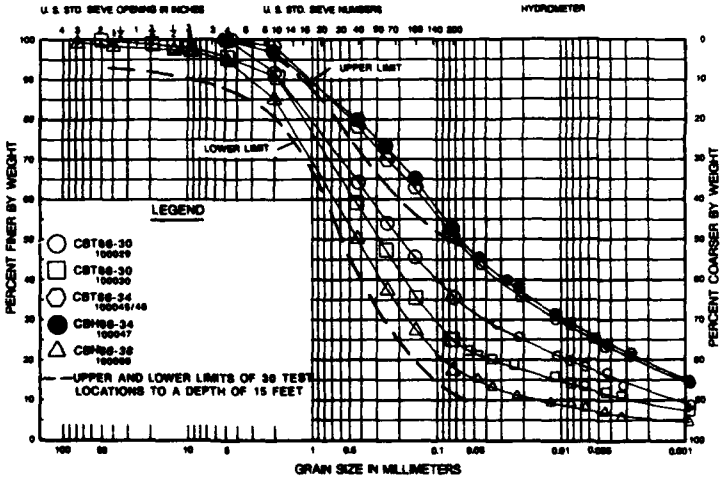
SUBMITTED BY:
DATE APPROVED:
SPEC. NO. DACW09-___-___
DISTRICT FILE NO.
SHEET

PLATE A-78



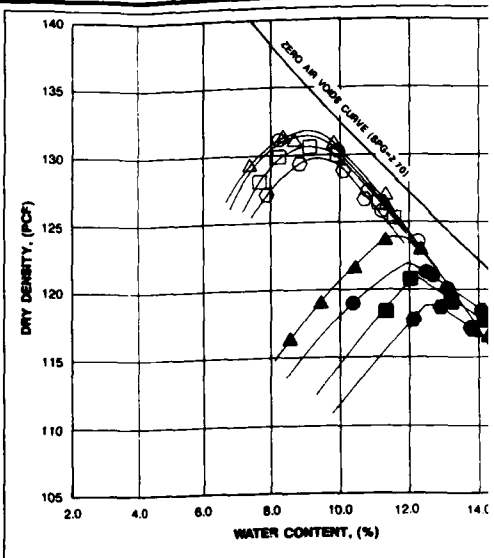
SYMBOL	HOLE/ SAMPLE NUMBER	ELEV./ DEPTH (FT)	OPT MOIST CONT (%)	MAX DRY DEN (PCF)	USCS CLASS	%> No 4 SIEVE	%< No.200 SIEVE	LL	PI	Sp. G.	REMARKS
○	CBT86-30 100029	0.75'-5'	15.4	109.3	CLAYEY SAND (SC)	0	36	35	15	2.69	3% BENTONITE(Na) ASTM D 698-78(A)
□	CBT86-30 100030	5.0'-11.0'	14.0	113.8	CLAYEY SAND (SC)	3	25	33	14	2.70	3% BENTONITE(Na) ASTM D 698-78(A)
⬡	CBT86-34 100045/46	0.75'-5'	13.7	113.4	SANDY CLAY (CL)	0	51	30	13	2.71	3% BENTONITE(Na) ASTM D 698-78(A)
■	CBT86-34 100045/46	0.75'-5'	13.2	114.3	SANDY CLAY (CL)	0	51	30	13	2.71	3% BENTONITE(Na) ASTM D 698-78(A)
●	CBH86-34 100047	27.5'-32'	14.7	114.6	SANDY CLAY (CL)	0	53	30	14	2.71	3% BENTONITE(Na) ASTM D 698-78(A)
△	CBH86-38 100069	7'-12'	13.2	117.4	CLAYEY SAND (SC)	5	17	29	10	2.71	3% BENTONITE(Na) ASTM D 698-78(A)

COMPACTION TEST DATA



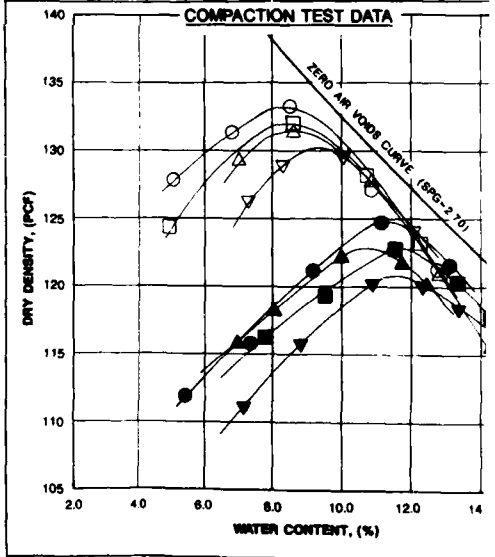
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

GRADATION CURVES



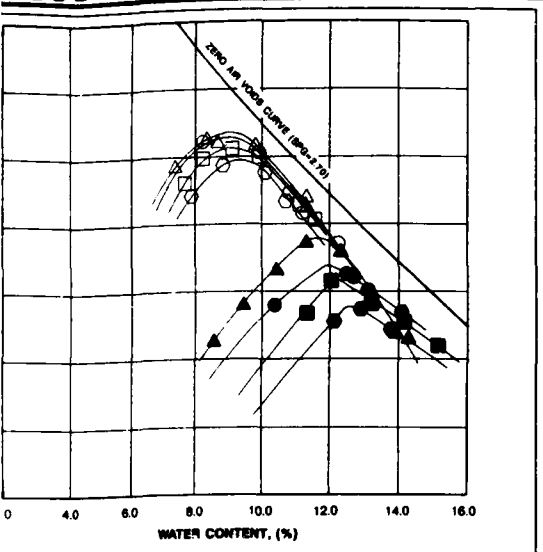
SYMBOL	HOLE/ SAMPLE NUMBER	ELEV./ DEPTH (FT)	OPT MOIST CONT (%)	MAX DRY DEN (PCF)	USCS CLASS	%> No 4 SIEVE	%< No.200 SIEVE	LL	PI	Sp
▲	SMI-1	N/A	11.7	123.9	CLAYEY SAND (SC-SM)	2	32	23	6	
△	SMI-1	N/A	9.0	131.6	CLAYEY SAND (SC-SM)	2	32	23	6	
●	SMI-2	N/A	11.9	121.9	CLAYEY SAND (SC)	2	36	27	10	
○	SMI-2	N/A	9.1	131.6	CLAYEY SAND (SC)	2	36	27	10	
■	SMI-3	N/A	12.2	120.8	CLAYEY SAND (SC)	2	33	30	13	
□	SMI-3	N/A	9.1	130.7	CLAYEY SAND (SC)	2	33	30	13	
●	SMI-4	N/A	12.6	118.9	CLAYEY SAND (SC)	2	36	33	15	
⬡	SMI-4	N/A	9.4	129.8	CLAYEY SAND (SC)	2	36	33	15	

COMPACTION TEST DATA



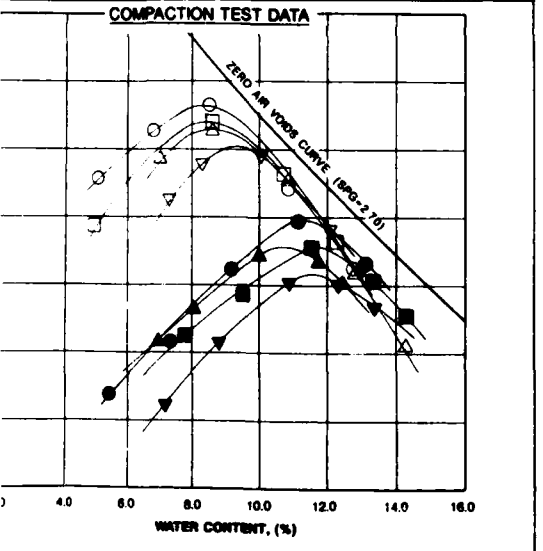
SYMBOL	HOLE/ SAMPLE NUMBER	ELEV./ DEPTH (FT)	OPT. MOIST. CONT (%)	MAX. DRY DEN. (PCF)	USCS CLASS.	%> No. 4 SIEVE	%< No.200 SIEVE	LL	PI	Sp
▲	SM-1	N/A	10.7	122.8	CLAYEY SAND (SC-SM)	1	35	23	4	
△	SM-1	N/A	8.6	131.5	CLAYEY SAND (SC-SM)	1	35	23	4	
●	SM-2	N/A	11.4	124.6	CLAYEY SAND (SC-SM)	1	34	25	9	
○	SM-2	N/A	8.3	133.1	CLAYEY SAND (SC-SM)	1	34	25	9	
■	SM-3	N/A	11.8	122.9	CLAYEY SAND (SC)	1	35	29	11	
□	SM-3	N/A	8.5	132.0	CLAYEY SAND (SC)	1	35	29	11	
▼	SM-4	N/A	11.8	120.8	CLAYEY SAND (SC)	1	35	32	15	
▽	SM-4	N/A	9.4	130.2	CLAYEY SAND (SC)	1	35	32	15	

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

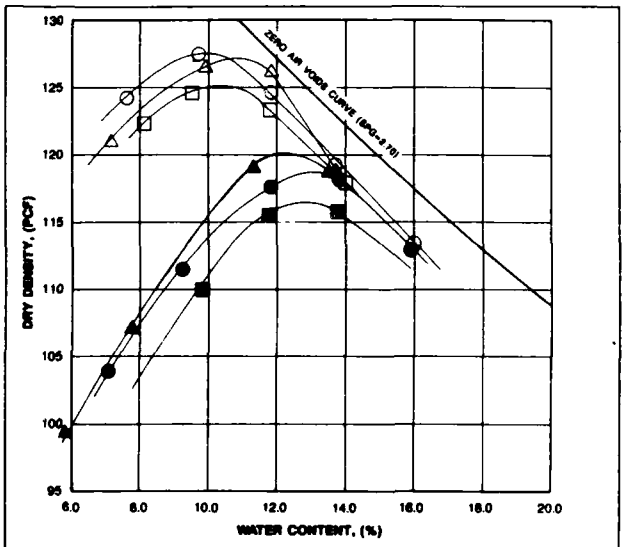


	ELEV./ DEPTH (FT)	OPT MOIST CONT (%)	MAX DRY DEN (PCF)	USCS CLASS	%> No 4 SIEVE	%< No 200 SIEVE	LL	PI	Sp G	REMARKS
1	N/A	11 7	123 9	CLAYEY SAND (SC-SM)	2	32	23	6		CONTROL SAMPLE ASTM D 698-78(C)
1	N/A	9 0	131 8	CLAYEY SAND (SC-SM)	2	32	23	6		CONTROL SAMPLE ASTM D 1557-78(C)
2	N, A	11 9	121 9	CLAYEY SAND (SC)	2	36	27	10		1% BENTONITE(Na) ASTM D 698-78(C)
2	N/A	9 1	131 6	CLAYEY SAND (SC)	2	36	27	10		1% BENTONITE(Na) ASTM D 1557-78(C)
3	N/A	12 2	120 8	CLAYEY SAND (SC)	2	33	30	13		3% BENTONITE(Na) ASTM D 698-78(C)
3	N/A	9 1	130 7	CLAYEY SAND (SC)	2	33	30	13		3% BENTONITE(Na) ASTM D 1557-78(C)
	N/A	12 6	118 9	CLAYEY SAND (SC)	2	36	33	15		5% BENTONITE(Na) ASTM D698-78(C)
	N/A	9 4	129 8	CLAYEY SAND (SC)	2	36	33	15		5% BENTONITE Na ASTM D 1557-78(C)

COMPACTION TEST DATA

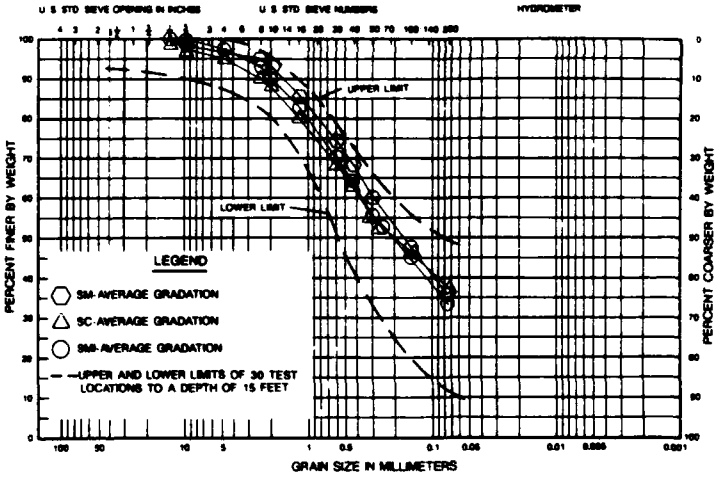


	ELEV./ DEPTH (FT)	OPT MOIST CONT (%)	MAX DRY DEN (PCF)	USCS CLASS	%> No 4 SIEVE	%< No 200 SIEVE	LL	PI	Sp. G.	REMARKS
	N/A	10 7	122 8	CLAYEY SAND (SC-SM)	1	35	23	4		CONTROL SAMPLE ASTM D 698-78(C)
	N/A	8 6	131 5	CLAYEY SAND (SC-SM)	1	35	23	4		CONTROL SAMPLE ASTM D 1557-78(C)
	N/A	11 4	124 6	CLAYEY SAND (SC-SM)	1	34	25	9		1% BENTONITE(Na) ASTM D 698-78(C)
	N/A	8 3	133 1	CLAYEY SAND (SC-SM)	1	34	25	9		1% BENTONITE(Na) ASTM D 1557-78(C)
	N/A	11 8	122 9	CLAYEY SAND (SC)	1	35	29	11		3% BENTONITE(Na) ASTM D 698-78(C)
	N/A	8 5	132 0	CLAYEY SAND (SC)	1	35	29	11		3% BENTONITE(Na) ASTM D 1557-78(C)
	N/A	11 6	120 8	CLAYEY SAND (SC)	1	35	32	15		5% BENTONITE(Na) ASTM D 698-78(C)
	N/A	9 4	130 2	CLAYEY SAND (SC)	1	35	32	15		5% BENTONITE(Na) ASTM D 1557-78(C)



SYMBOL	HOLE/ SAMPLE NUMBER	ELEV./ DEPTH (FT)	OPT MOIST CONT (%)	MAX DRY DEN (PCF)	USCS CLASS	%> No 4 SIEVE	%< No 200 SIEVE	LL	PI	Sp G	REMARKS
▲	SC-1	N/A	12 2	120 1	CLAYEY SAND (SC)	4	38	32	16		CONTROL SAMPLE ASTM D 698-78(C)
△	SC-1	N/A	10 8	127 2	CLAYEY SAND (SC)	4	38	32	16		CONTROL SAMPLE ASTM D 1557-78(C)
●	SC-2	N/A	13 1	118 8	CLAYEY SAND (SC)	4	38	34	16		1% BENTONITE(Na) ASTM D 698-78(C)
○	SC-2	N/A	10 0	127 5	CLAYEY SAND (SC)	4	38	34	16		1% BENTONITE(Na) ASTM D 1557-78(C)
■	SC-3	N/A	12 8	116 5	CLAYEY SAND (SC)	4	38	39	21		3% BENTONITE(Na) ASTM D 698-78(C)
□	SC-3	N/A	10 3	125 1	CLAYEY SAND (SC)	4	38	39	21		3% BENTONITE(Na) ASTM D 1557-78(C)

COMPACTION TEST DATA



GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

GRADATION CURVES

NOTES:

1. REFER TO PLATE A-60 FOR LOCATION OF TEST HOLES.
2. REFER TO PLATES A-62 THRU A-72 FOR SOIL LOGS.
3. TEST METHOD ASTM D 1557-78(METHOD C) WAS FOLLOWED WITH THE EXCEPTION THAT A MECHANICAL RAMMER HAVING A CIRCULAR CONTACT FACE TWO INCHES IN DIAMETER WAS USED WITH THE SIX INCH DIAMETER COMPACTION MOLD.

SYMBOL	DESCRIPTION	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

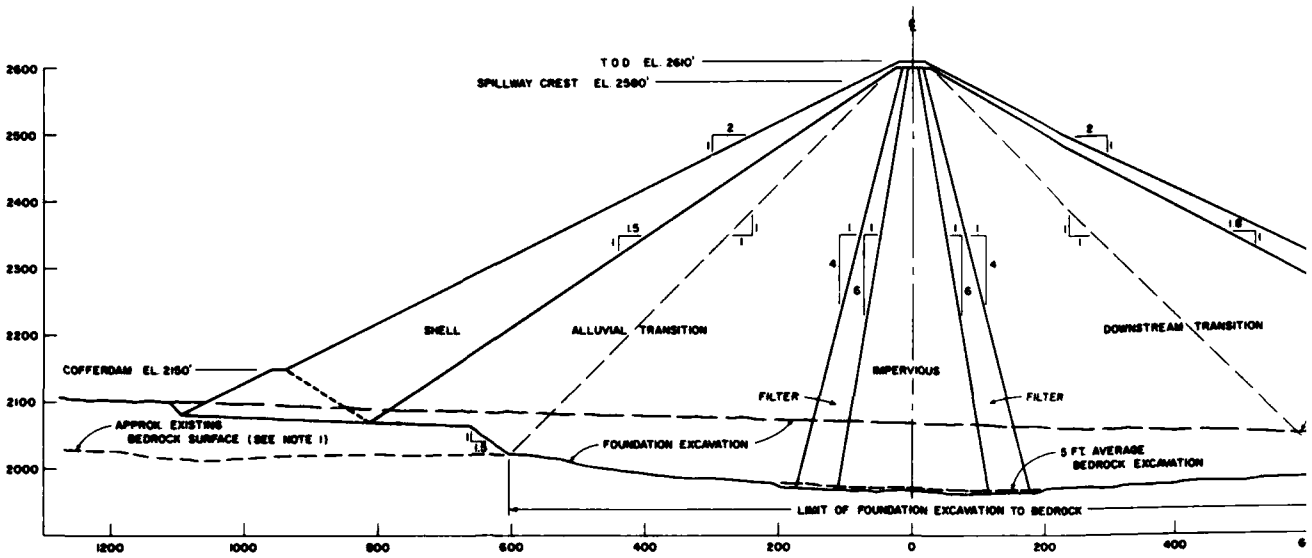
SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM
IMPERVIOUS BORROW AREA
MOISTURE-DENSITY RELATIONSHIPS
(BENTONITE AMENDED)

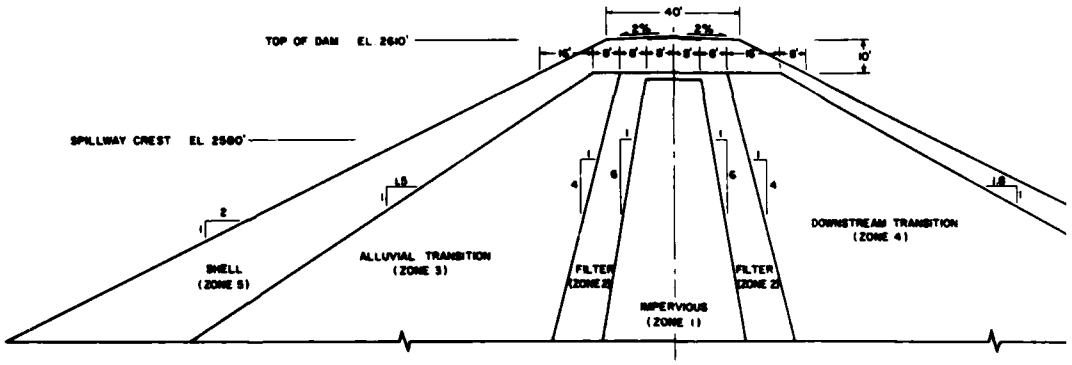
RECOMMENDED BY:		
DRAWN BY:		
CHECKED BY:		
SUBMITTED BY:	DATE APPROVED:	SHEET
CHIEF, ENGINEERING DIVISION		

SPEC. NO. DACW09-____-B-____
DISTRICT FILE NO.

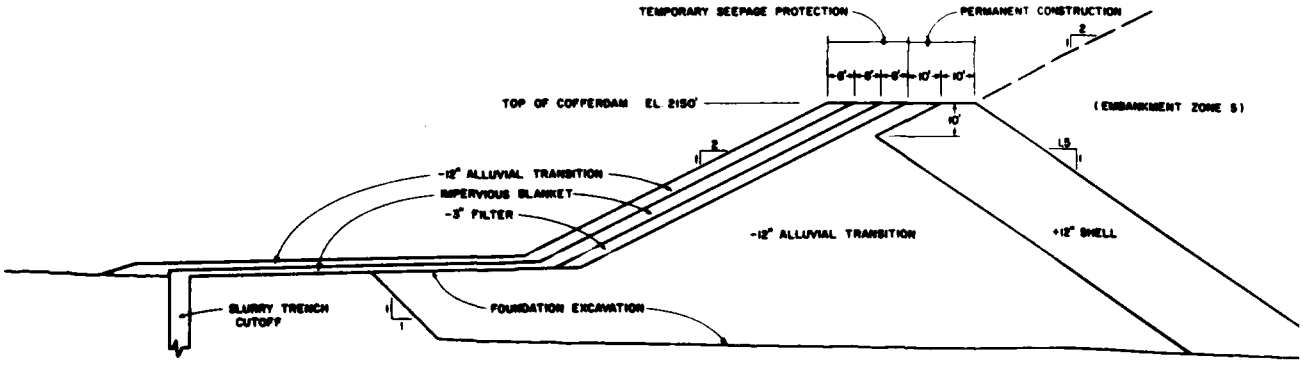
PLATE A-79



MAXIMUM EMBANKMENT SECTION
SCALE 1 IN. = 100 FT

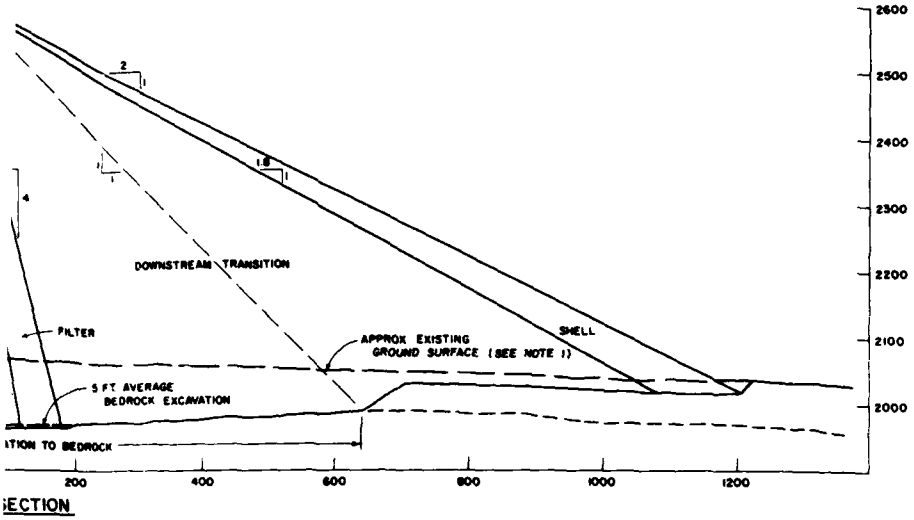


CROWN DETAIL
SCALE 1 IN. = 20 FT

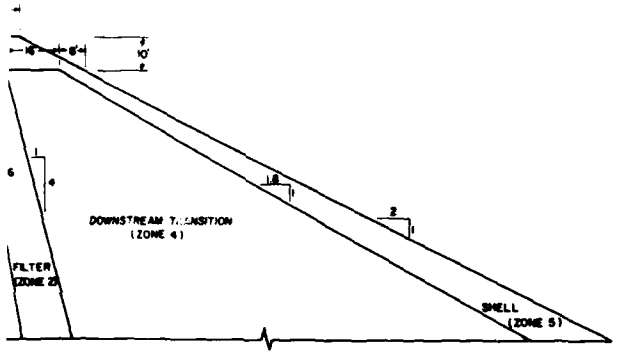


COFFERDAM DETAIL
SCALE: 1 IN. = 20 FT

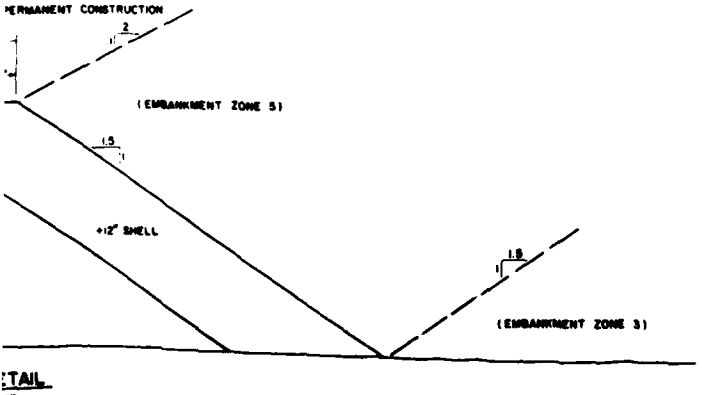
ALUE ENGINEERING PAYS



ECTION



L
T



TAIL
T

LEGEND:

ZONE 1 — IMPERVIOUS — CLAYEY SANDS AND SILTY SANDS FROM IMPERVIOUS BORROW AREA

ZONE 2 — FILTER — MINUS 3-INCH SAND AND GRAVEL PROCESSED FROM EXCAVATION OF ALLUVIAL MATERIALS IN FOUNDATION AND DOWNSTREAM PERVIOUS BORROW AREA.

ZONE 3 — ALLUVIAL TRANSITION — MINUS 12-INCH SAND, GRAVEL AND COBBLES PROCESSED FROM EXCAVATION OF ALLUVIAL MATERIALS IN FOUNDATION AND DOWNSTREAM PERVIOUS BORROW AREA.

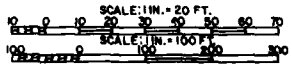
ZONE 4 — DOWNSTREAM TRANSITION — SAND SIZE TO APPROXIMATELY 9-INCH ROCK FRAGMENTS FROM SPILLWAY AND DRAINAGE CHANNEL EXCAVATION.

ZONE 5 — SHELL — 12-INCH TO 30-INCH COBBLES AND BOULDERS PROCESSED FROM EXCAVATION OF ALLUVIAL MATERIALS IN FOUNDATION AND DOWNSTREAM PERVIOUS BORROW AREA, WITH INTERMEDIATE TRAFFIC LAYERS OF ZONE 3 MATERIAL.

NOTES

1 MAXIMUM EMBANKMENT SECTION SHOWN IS DRAWN APPROXIMATELY THROUGH A LINE ALONG THE CENTER OF THE CANYON, AND IS FOR ILLUSTRATIVE PURPOSES ONLY FOR ACTUAL SECTIONS, SEE MAIN REPORT

2



DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

SYMBOL	DESCRIPTION	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

EMBANKMENT SECTION AND DETAILS

SUBMITTED BY: DATE APPROVED:
CHIEF, ENGINEERING DIVISION

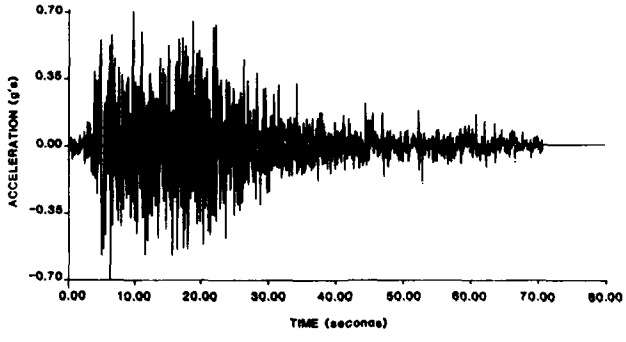
SPEC. NO. DACW09- - -
DISTRICT FILE NO.

SHEET

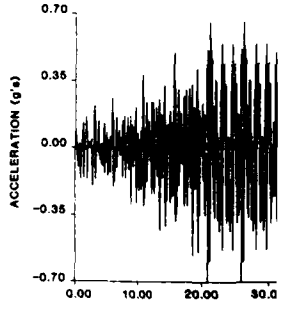
SAFETY PAYS

PLATE A-89

ENVIRONMENTAL
ENHANCEMENT
THRU ENGINEERING



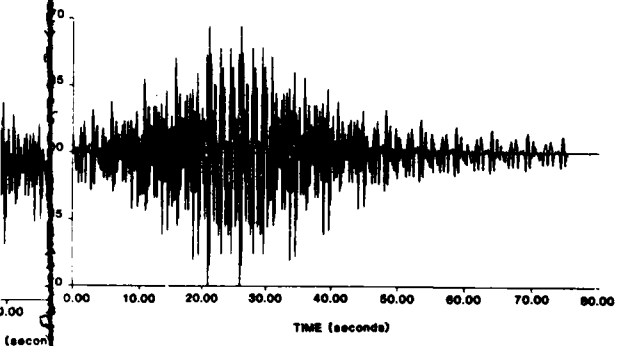
Bolt 8+ Magnitude Earthquake Record Scaled to 0.70g Max.



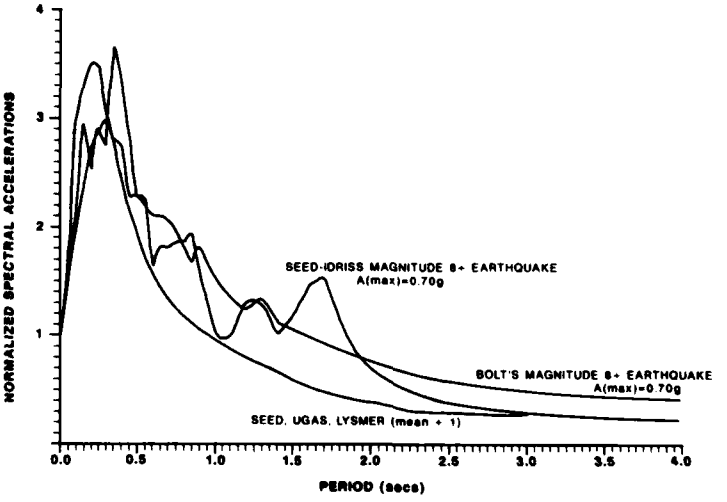
Seed-Idriss 8+ Magnitude E

1 RESPONSE SPECTRA FOR BASE ROCK M
THE RESPONSE SPECTRA WERE NORMAL

2 THE BOLT 8+ MAGNITUDE EARTHQUAKE
ANALYSIS OF SAN ANDREAS AND PILAC



Seed-Idriss 8+ Magnitude Earthquake Record Scaled to 0.70g Max.



NORMALIZED ACCELERATION SPECTRA
FOR BASE ROCK MOTIONS

NSE SPECTRA FOR BASE ROCK MOTIONS WERE DETERMINED USING THE 'SHAKE' COMPUTER PROGRAM WITH A VALUE OF 5% DAMPING.
ESPONSE SPECTRA WERE NORMALIZED TO A MAXIMUM GROUND ACCELERATION VALUE OF 0.70 g's. (BOLT & SEED-IDRISS RECORDS)
OLT 8+ MAGNITUDE EARTHQUAKE RECORD WAS DEVELOPED BY ESA WITH CONSULTATION BY B. BOLT FOR THE SEISMIC STABILITY
YSIS OF SAN ANDREAS AND PILACIRTOS DAMS.

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

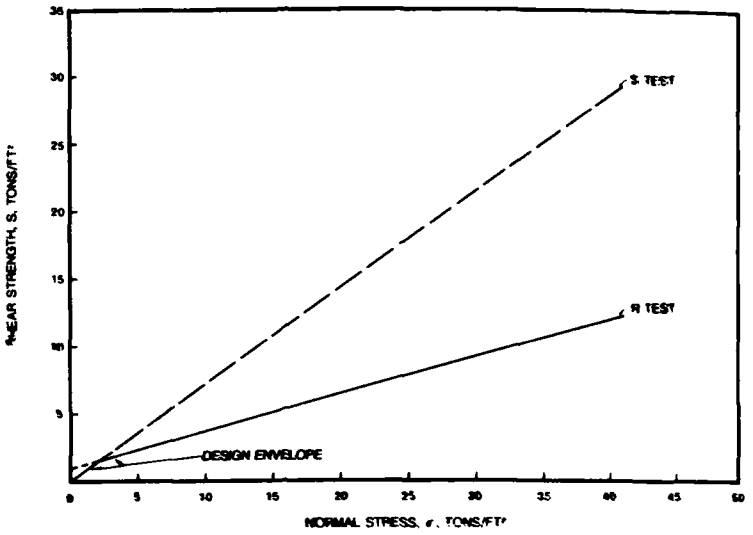
SEVEN OAKS DAM
EARTHQUAKE ACCELEROGRAPH RECORDS
AND
RESPONSE SPECTRUMS

SUBMITTED BY: DATE APPROVED:
CHIEF, ENGINEERING DIVISION

SPEC. NO. DACW09-____-8-____
DISTRICT FILE NO.

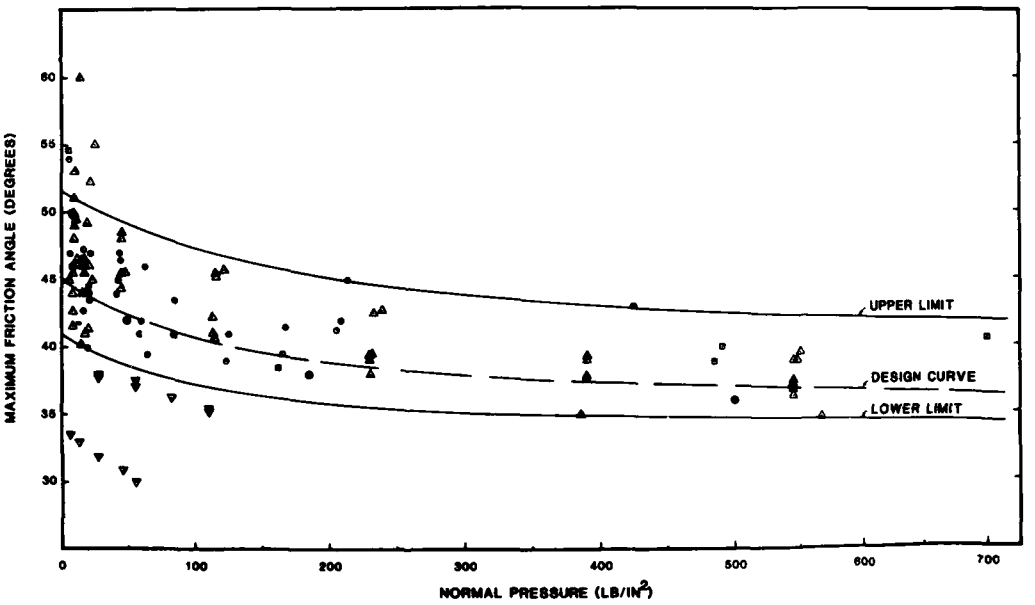
SHEET

PLATE A-81



SHEAR STRENGTH OF IMPERVIOUS MATERIALS (ZONE 1)

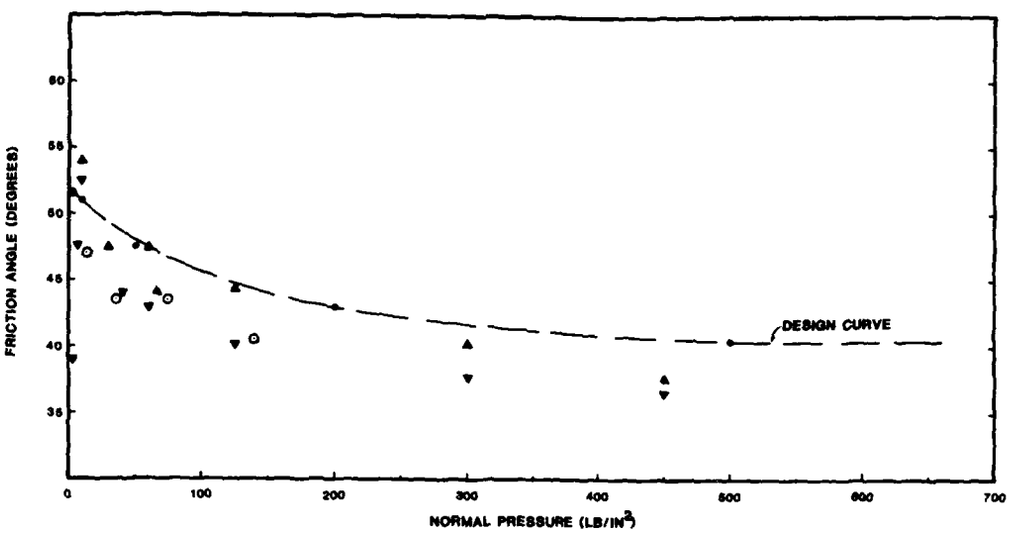
ENVIRONMENTAL
ENHANCEMENT
THRU ENGINEERING



LEGEND:

SYMBOL	MAXIMUM PARTICLE SIZE
▽	Nos. 20, 14, 8
▫	3/4, 1-1/2 inches
●	3, 4 inches
△	7, 8 inches
◦	RECOMMENDED BY TECHNICAL EXPERT

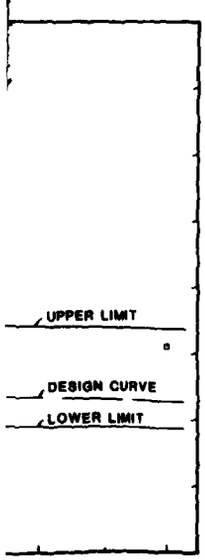
SHEAR STRENGTH OF DOWNSTREAM TRANSITION MATERIALS (ZONE 4)



LEGEND:

- ○ TRIAXIAL TEST RESULTS (MINUS 3-INCH MATERIALS)
- • RECOMMENDED BY TECHNICAL EXPERT
- ▲ HIGH DENSITY } TEST RESULTS FROM OTHER PROJECTS
- ▼ MEDIUM DENSITY }

SHEAR STRENGTH OF ALLUVIAL TRANSITION AND SHELL MATERIALS (ZONES 3 & 5)



SYMBOL	DESCRIPTION	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

EMBANKMENT SLOPE STABILITY ANALYSIS
DESIGN CURVES

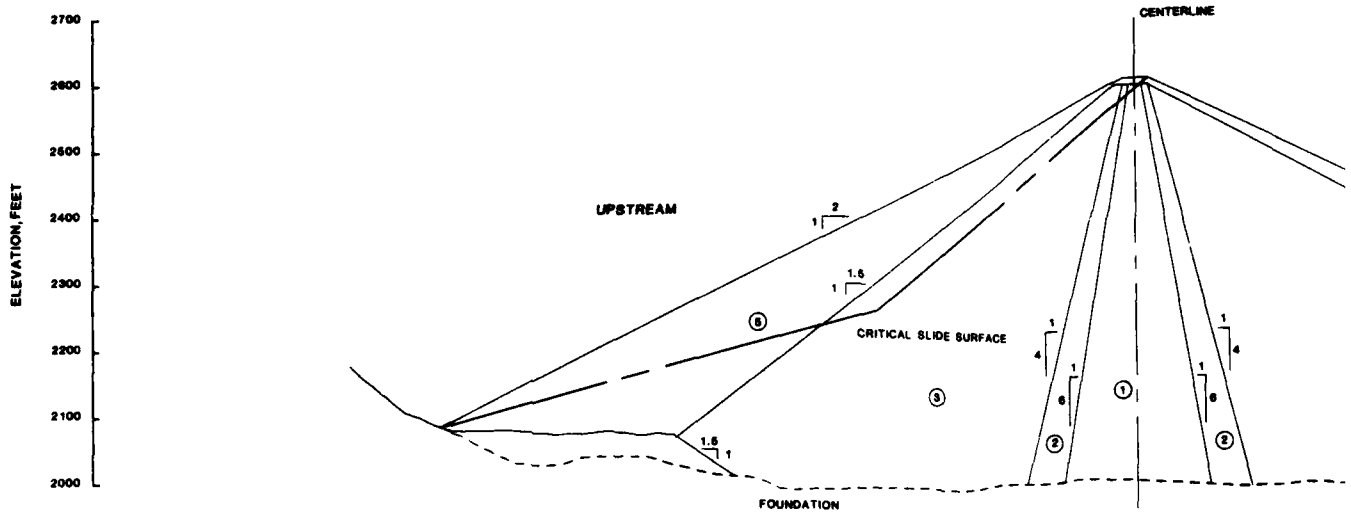
SUBMITTED BY: DATE APPROVED:

SPEC. NO. SACW09- ____ -____

DISTRICT FILE NO.

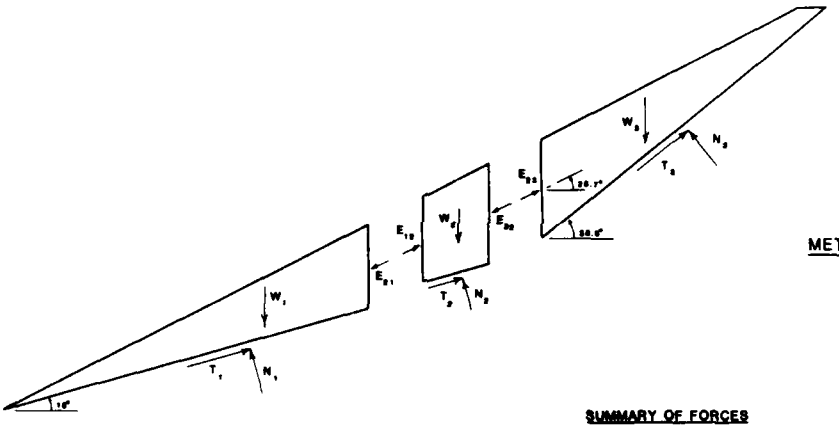
SHEET

PLATE A-82

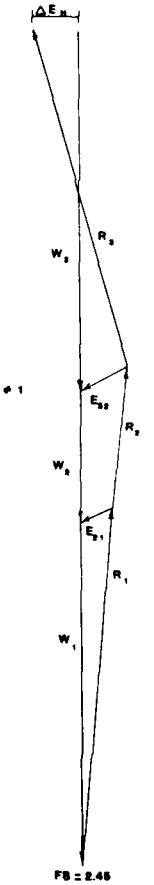


EMBANKMENT SECTION

SCALE 1 INCH : 100 FEET



METHOD OF ANALYSIS



SUMMARY OF FORCES

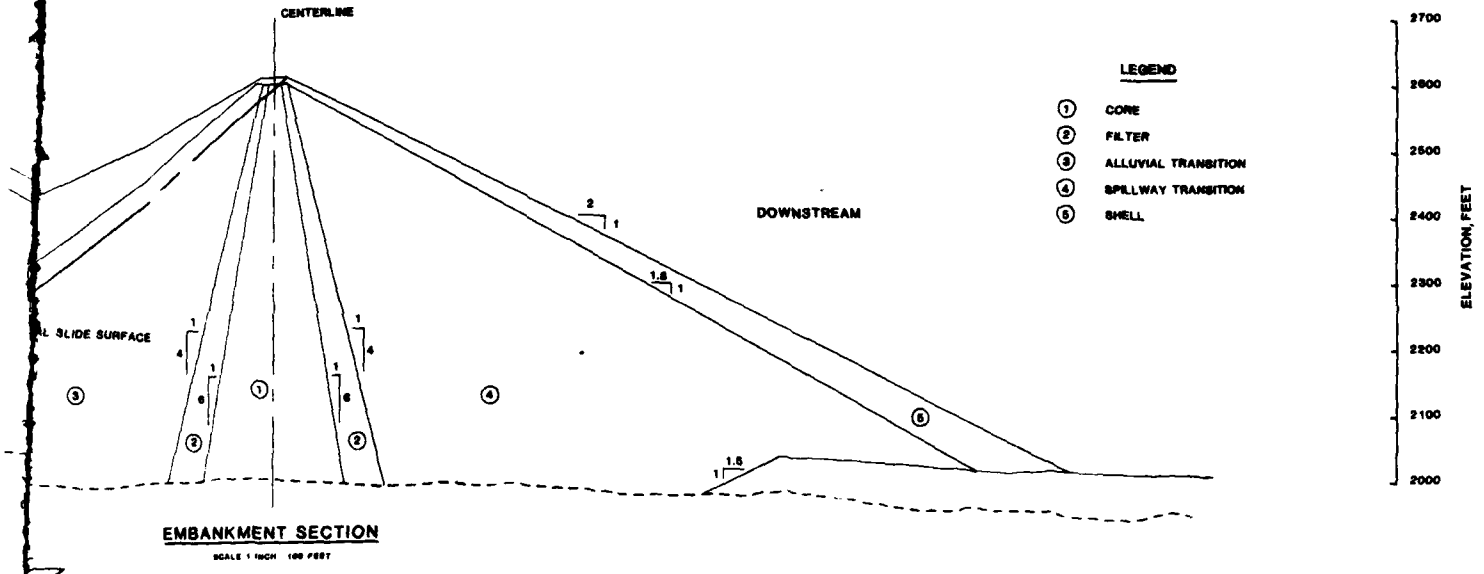
	WEDGE/ BLOCK	WEIGHT (W) (KIPS)	TANGENT (T) (KIPS)	NORMAL (N) (KIPS)	RESULTANT (R) (KIPS)	RESULTANT (R) (DEG)	SIDE FORCE (E) (KIPS)	SIDE FORCE (DEG)
TRIAL #1	1	5177	1843	5098	5421	85.1	517	25.7
	2	1992	787	1976	2127	83.2	794	25.7
	3	5389	1999	4885	5278	106.2		
TRIAL #2	1	5177	2723	5396	6044	78.2	1374	25.7
	2	1992	883	1994	2181	81.1	1750	25.7
	3	5389	1988	4334	4768	103.9		

NOTES :

1. R : RESULTANT FORCE OF T AND N FORCES
2. RESULTANT ANGLES ARE MEASURED FROM THE (+) HORIZONTAL AXIS.
3. HORIZONTAL WATER FORCES ARE AT AN EQUILIBRIUM STATE IN SUDDEN DRAWDOWN AND PARTIAL POOL CONDITIONS.
4. SIDE FORCE ANGLE WAS OBTAINED FROM COMPUTER RUN OF UTEXAS2, SPENCER'S METHOD.

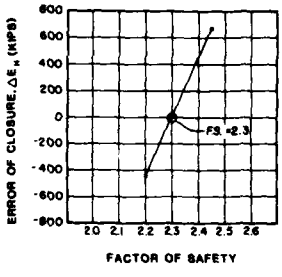
ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

VALUE ENGINEERING PAYS

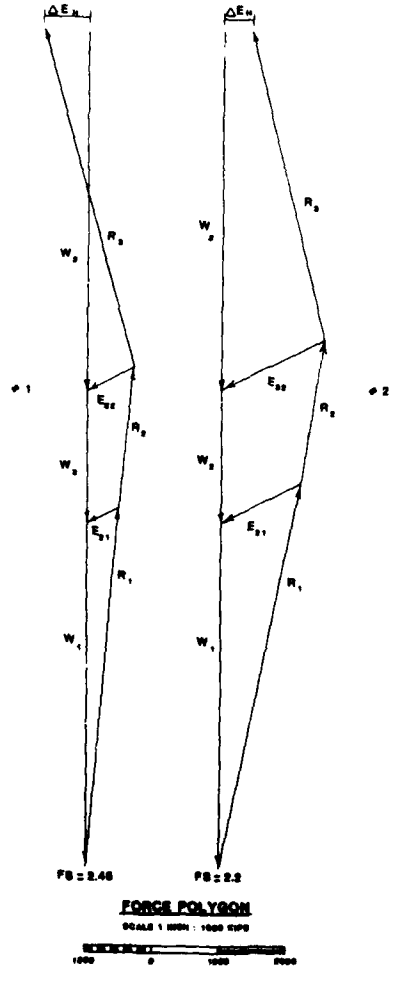


EMBANKMENT SECTION

SCALE 1 INCH 100 FEET



METHOD OF ANALYSIS



FORCE POLYGON

SCALE 1 INCH = 1000 KIPS

SIDE FORCE (E) (KIPS)	(DEG)
517	25.7
794	25.7
1376	25.7
1750	25.7

(+) HORIZONTAL AXIS.

RRUM STATE
DITIONS.

MPUTER RUN

GENERAL NOTES:

1. TRIAL WEDGES WERE ANALYZED AND CHECKED BY HAND USING THE GRAPHICAL WEDGE METHOD IN ACCORDANCE WITH EM 1110-2-1902.
2. FOUNDATION SHEAR STRENGTH EXCEEDS EMBANKMENT SHEAR STRENGTH.

DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

SYMBOL	DESCRIPTION	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

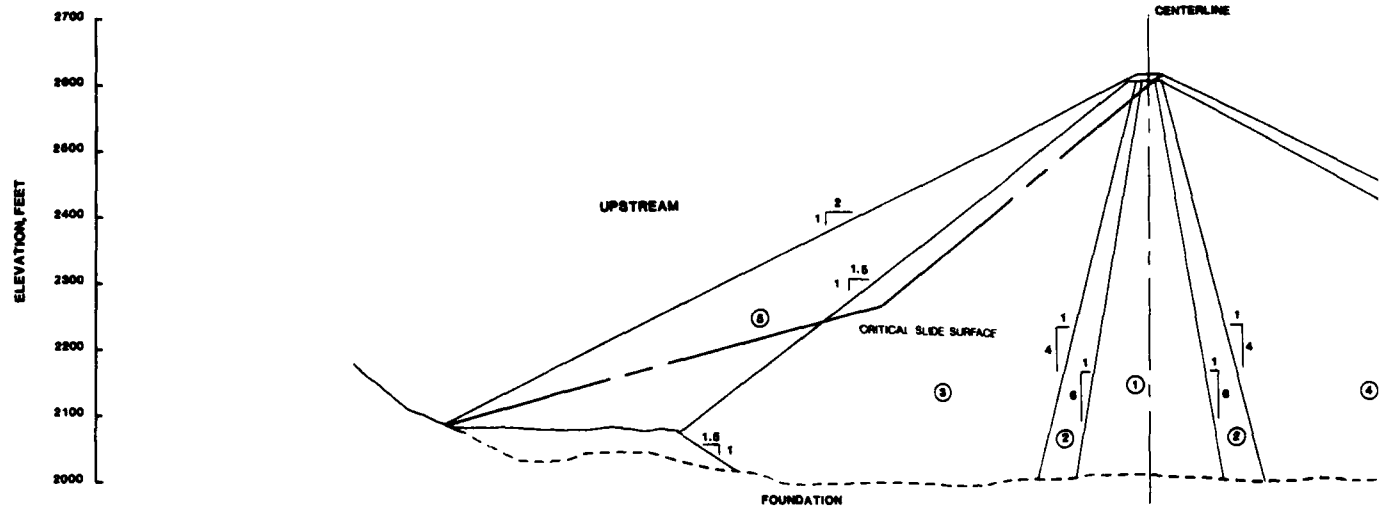
SEVEN OAKS DAM

EMBANKMENT SLOPE STABILITY ANALYSIS
END OF CONSTRUCTION
UPSTREAM SLOPE

SUBMITTED BY:
DATE APPROVED:
SPEC. NO. DACW09- -B-
DISTRICT FILE NO.
SHEET

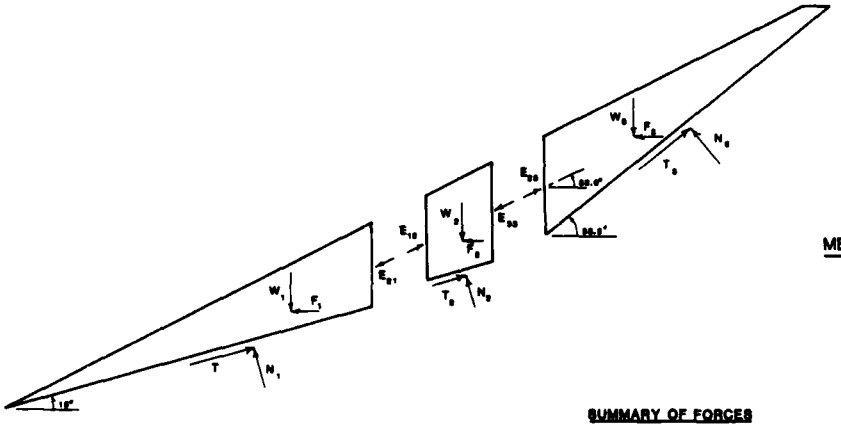
SAFETY PAYS

PLATE A-83

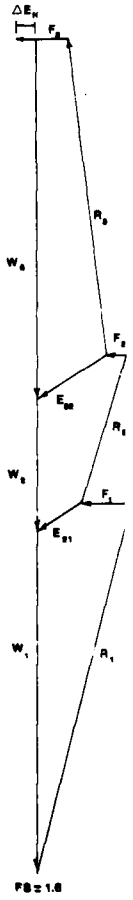


EMBANKMENT SECTION

SCALE 1 INCH = 100 FEET



METHOD OF ANALYSIS



FORCI

SCALE 1

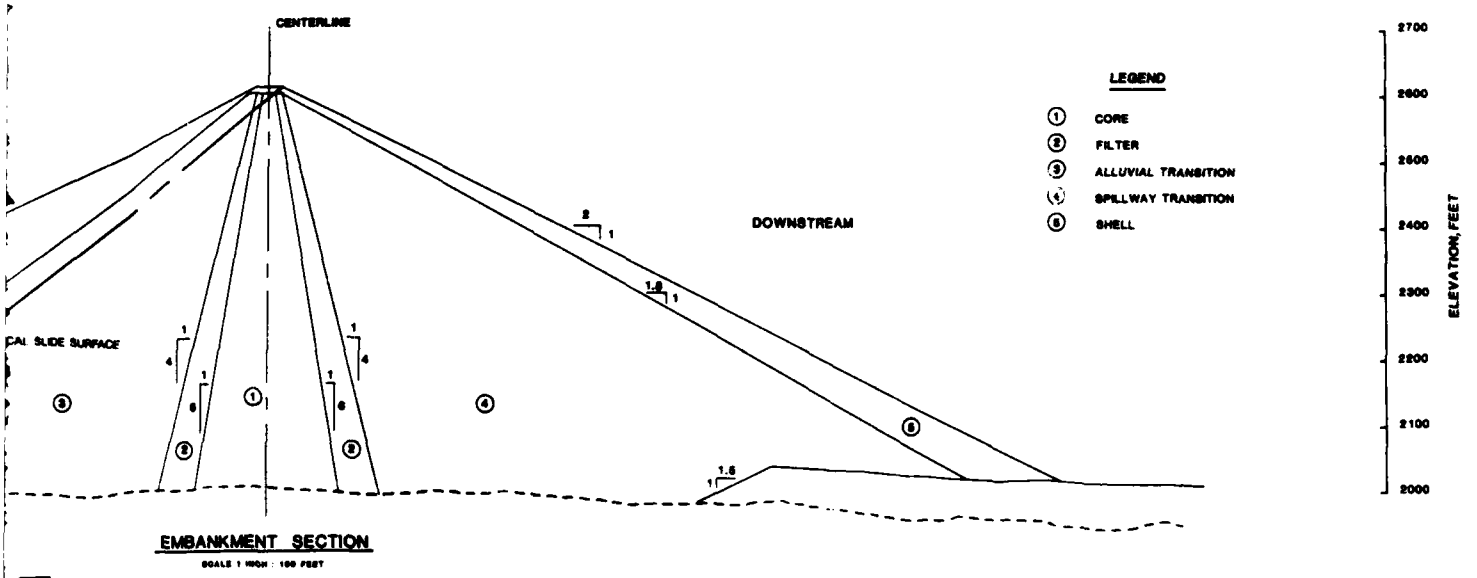
1000 0

SUMMARY OF FORCES

	WEDGE/ BLOCK	WEIGHT (W) (KIPS)	SEISMIC (F) (KIPS)	TANGENT (T) (KIPS)	NORMAL (N) (KIPS)	RESULTANT (R) (KIPS)	(DEG)	SIDE FORCE (E) (KIPS)	(DEG)
TRIAL Φ1	1	5177	776	2779	5026	5744	76.0	729	33.0
	2	1991	299	1204	1978	2315	73.6	1151	33.0
	3	5389	808	2556	4052	4791	98.3		
TRIAL Φ2	1	5177	776	3279	5189	6138	72.7	1254	33.0
	2	1991	299	1426	2050	2498	70.5	1910	33.0
	3	5389	808	2482	3531	4316	93.4		

NOTES : SEE NOTES ON PLATE A-76

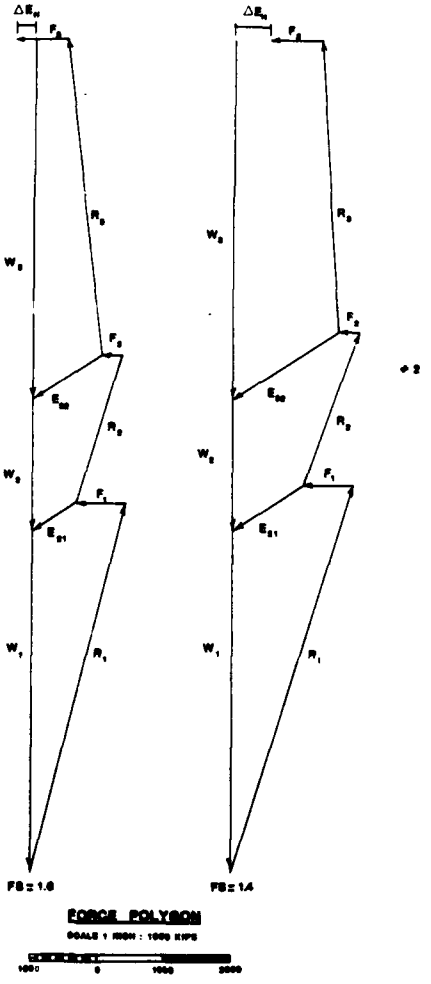
ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

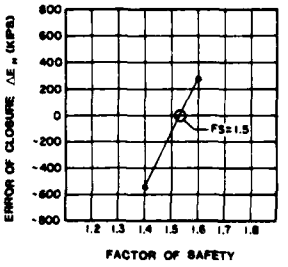


EMBANKMENT SECTION

SCALE 1 INCH : 100 FEET

METHOD OF ANALYSIS





SIDE FORCE (E) (KIPS)	(DEG)
729	33.0
1151	33.0
1254	33.0
1910	33.0

GENERAL NOTES :

1. TRIAL WEDGES WERE ANALYZED AND CHECKED BY HAND USING THE GRAPHICAL WEDGE METHOD IN ACCORDANCE WITH EM 1110-2-1902.
2. FOUNDATION SHEAR STRENGTH EXCEEDS EMBANKMENT SHEAR STRENGTH.
3. SEISMIC COEFFICIENT OF 0.15 FOR ZONE 4.

DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
REVISIONS			

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM
EMBANKMENT SLOPE STABILITY ANALYSIS
END OF CONSTRUCTION
UPSTREAM SLOPE WITH EARTHQUAKE

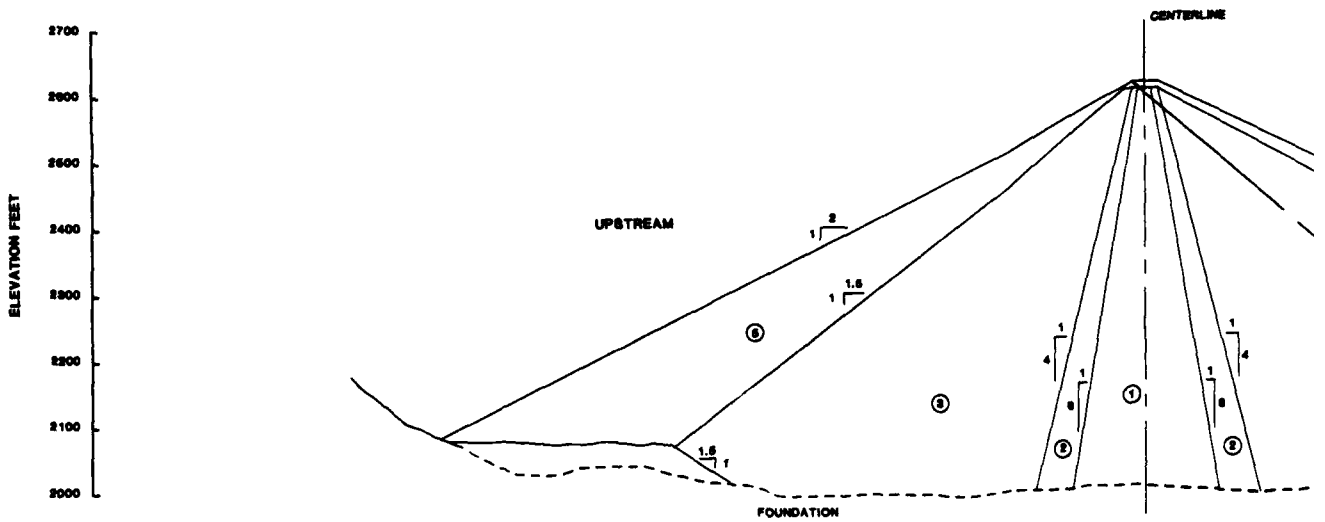
SUBMITTED BY: DATE APPROVED:

SPEC. NO. DACW09-____-____

DISTRICT FILE NO.

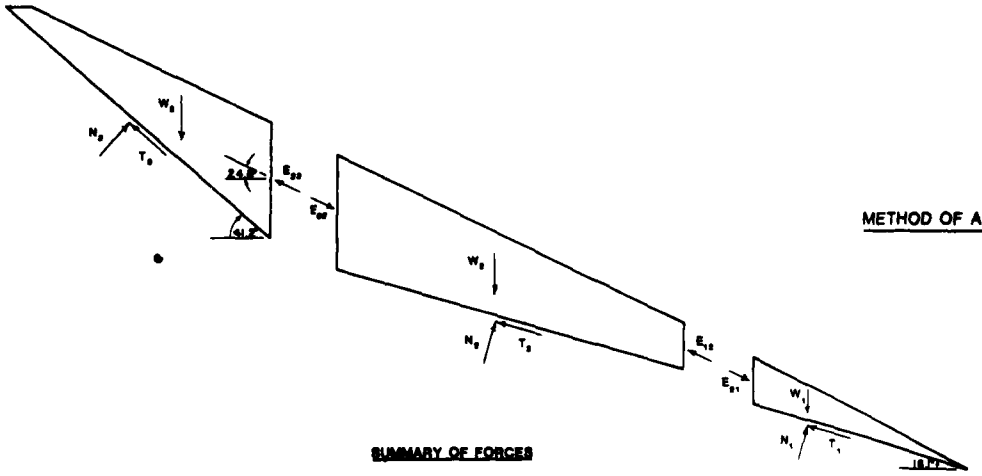
SHEET

PLATE A-84



EMBANKMENT SECTION

SCALE 1 INCH = 100 FEET



METHOD OF ANALYSIS

SUMMARY OF FORCES

	WEDGE/ BLOCK	WEIGHT (W) (KIPS)	TANGENT (T) (KIPS)	NORMAL (N) (KIPS)	RESULTANT (R) (KIPS)	(DEG)	SIDE FORCE (E) (KIPS)	(DEG)
TRIAL Φ 1	1	1715	993	1726	1991	103.8	522	24.8
	2	8703	3681	8549	9296	97.0	1771	24.8
	3	4682	1633	3807	4142	72.7		
TRIAL Φ 2	1	1715	895	1711	1931	101.5	423	24.8
	2	8703	3298	8495	9113	95.1	1315	24.8
	3	4682	1586	4093	4393	70.1		

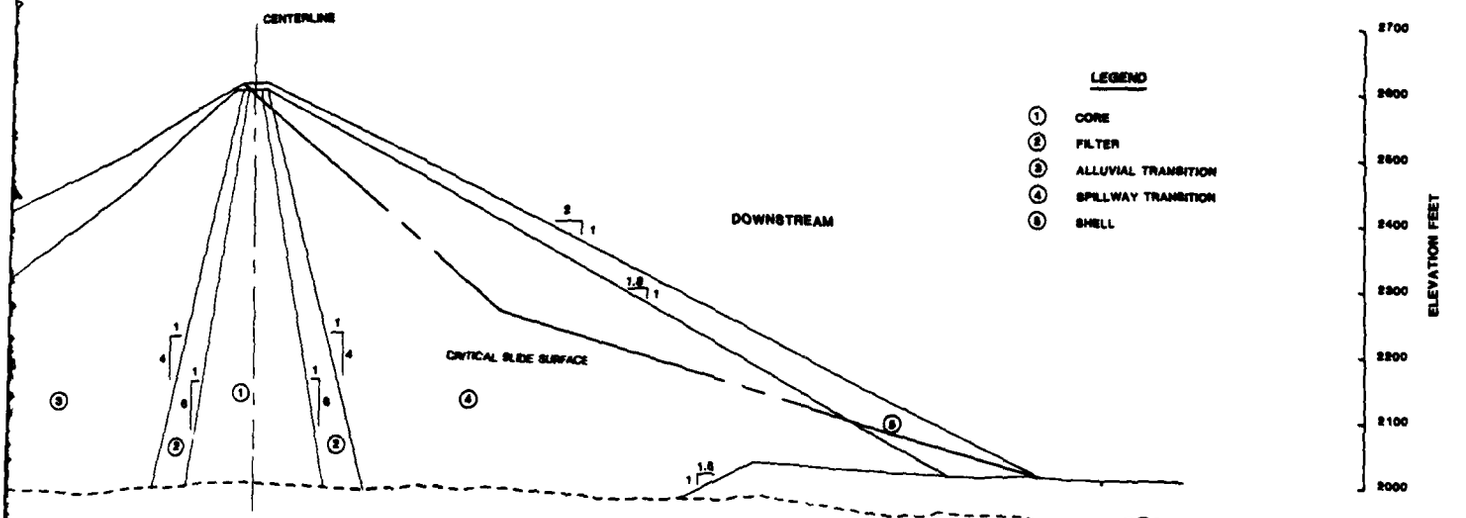
NOTES: SEE NOTES ON PLATE A-76

FORCE POLYGON

SCALE 1 INCH = 1000 KIPS

1000 0 1000 2

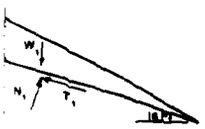
ENVIRONMENTAL
ENHANCEMENT
THRU ENGINEERING

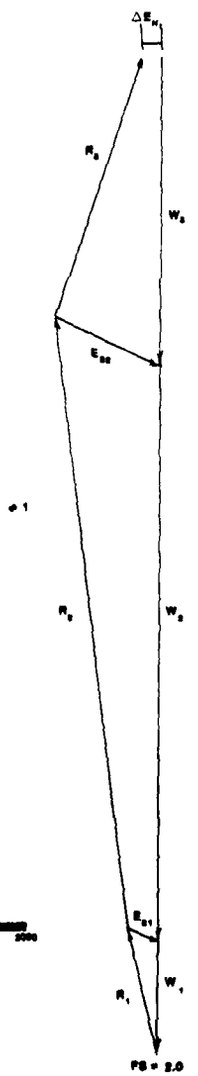


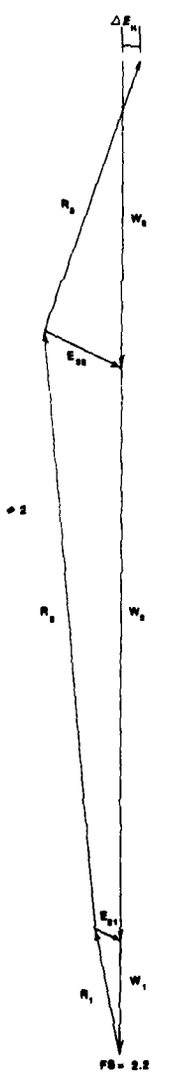
EMBANKMENT SECTION

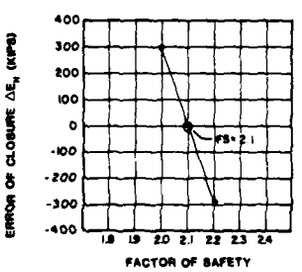
SCALE 1 INCH = 100 FEET

METHOD OF ANALYSIS









FORCE POLYGON

SCALE 1 INCH = 1000 KIPS

1000 0 1000 2000

GENERAL NOTES :

1. TRIAL WEDGES WERE ANALYZED AND CHECKED BY HAND USING THE GRAPHICAL WEDGE METHOD IN ACCORDANCE WITH EM 1110-2-1902.
2. FOUNDATION SHEAR STRENGTH EXCEEDS EMBANKMENT SHEAR STRENGTH.

DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

SYMBOL	DESCRIPTION	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

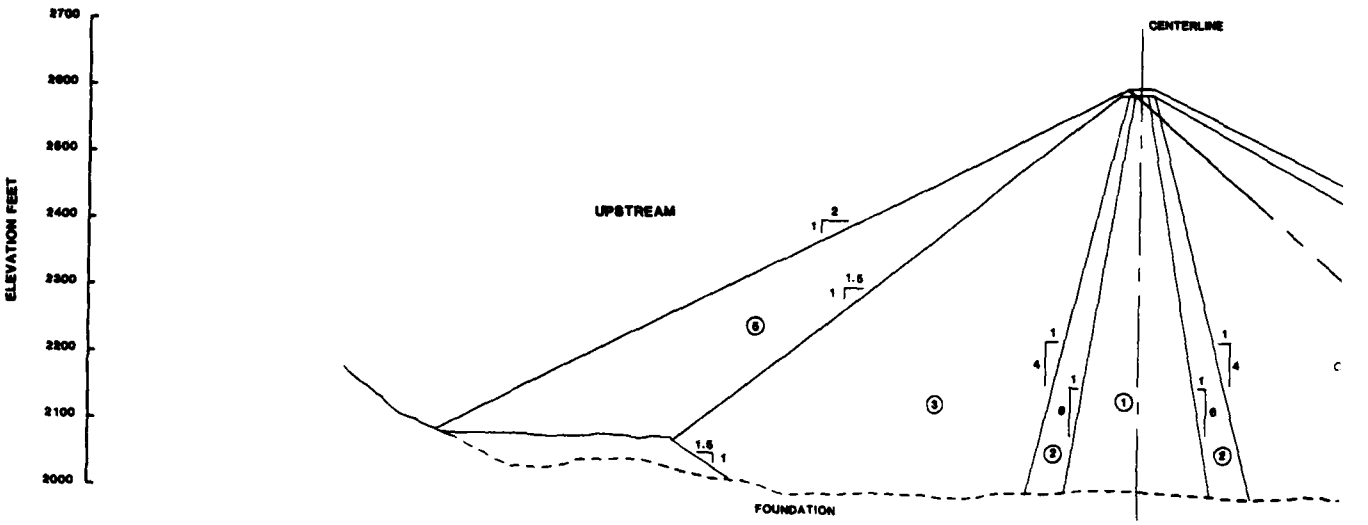
DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM
SEVEN OAKS DAM
EMBANKMENT SLOPE STABILITY ANALYSIS
END OF CONSTRUCTION
DOWNSTREAM SLOPE

SUBMITTED BY: DATE APPROVED:
CHIEF, DESIGN BRANCH

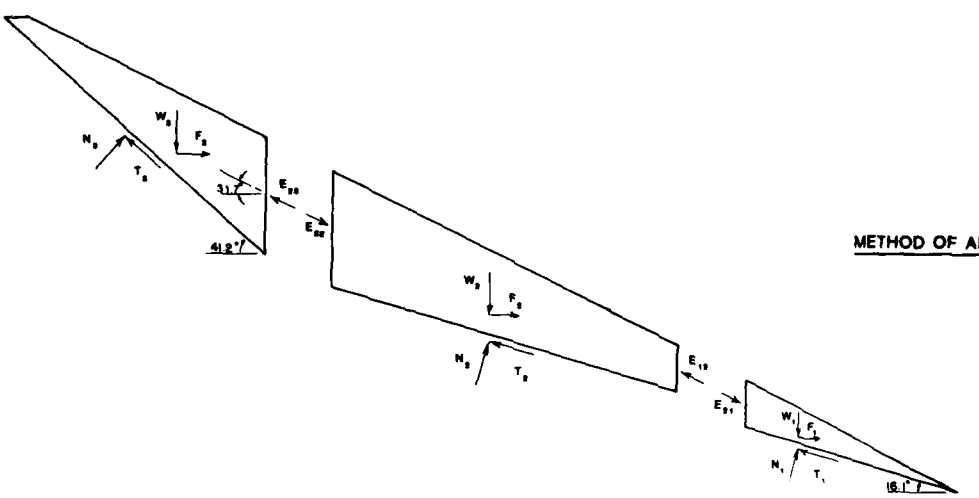
SPEC. NO. DACW09-__-__-____
DISTRICT FILE NO.
SHEET

PLATE A-66



EMBANKMENT SECTION

SCALE 1 INCH = 100 FEET



METHOD OF ANALYSIS



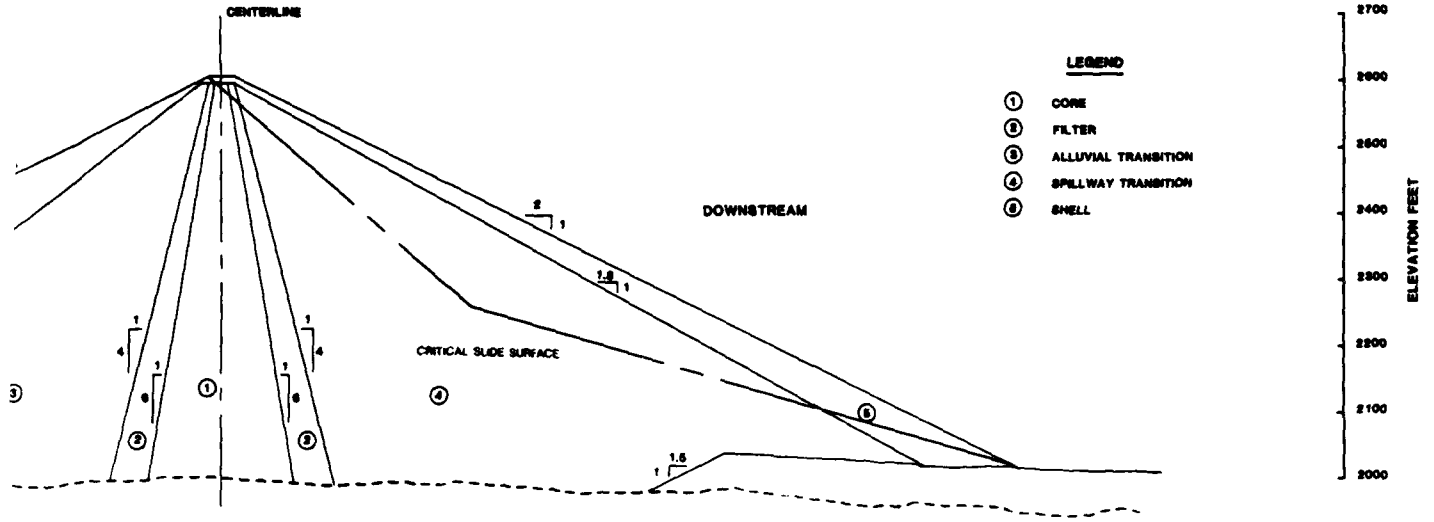
SUMMARY OF FORCES

	WEDGE/ BLOCK	WEIGHT (W) (KIPS)	SEISMIC (F) (KIPS)	TANGENT (T) (KIPS)	NORMAL (N) (KIPS)	RESULTANT (R) (KIPS)	RESULTANT (DEG)	SIDE FORCE (E) (KIPS)	SIDE FORCE (DEG)
TRIAL # 1	1	1715	257	1340	1747	2202	111.4	640	31.7
	2	8703	1306	4720	8291	9540	103.5	1730	31.7
	3	4682	702	1912	3342	3850	78.6		

NOTES: SEE NOTES ON PLATE A-76

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

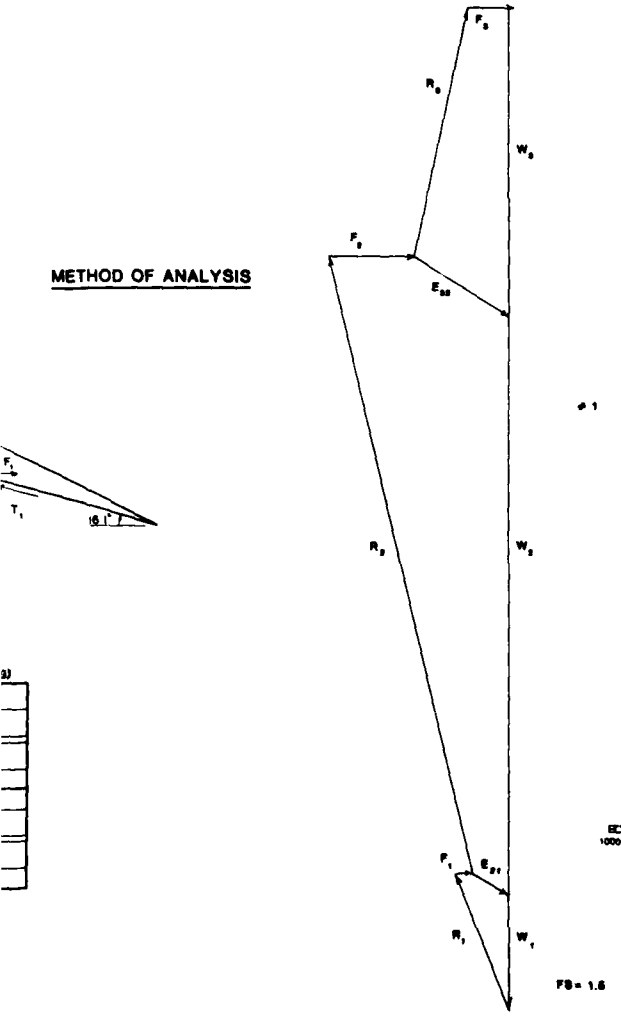
ALUE ENGINEERING PAYS

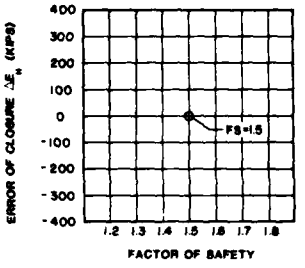


EMBANKMENT SECTION

SCALE 1 INCH : 100 FEET

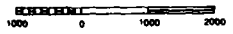
METHOD OF ANALYSIS





FORCE POLYGON

SCALE 1 INCH : 1000 KIPS



GENERAL NOTES :

1. TRIAL WEDGES WERE ANALYZED AND CHECKED BY HAND USING THE GRAPHICAL WEDGE METHOD IN ACCORDANCE WITH EM 1110-2-1902.
2. FOUNDATION SHEAR STRENGTH EXCEEDS EMBANKMENT SHEAR STRENGTH.
3. SEISMIC COEFFICIENT OF 0.15 FOR ZONE 4.

DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM
EMBANKMENT SLOPE STABILITY ANALYSIS
END OF CONSTRUCTION
DOWNSTREAM SLOPE WITH EARTHQUAKE

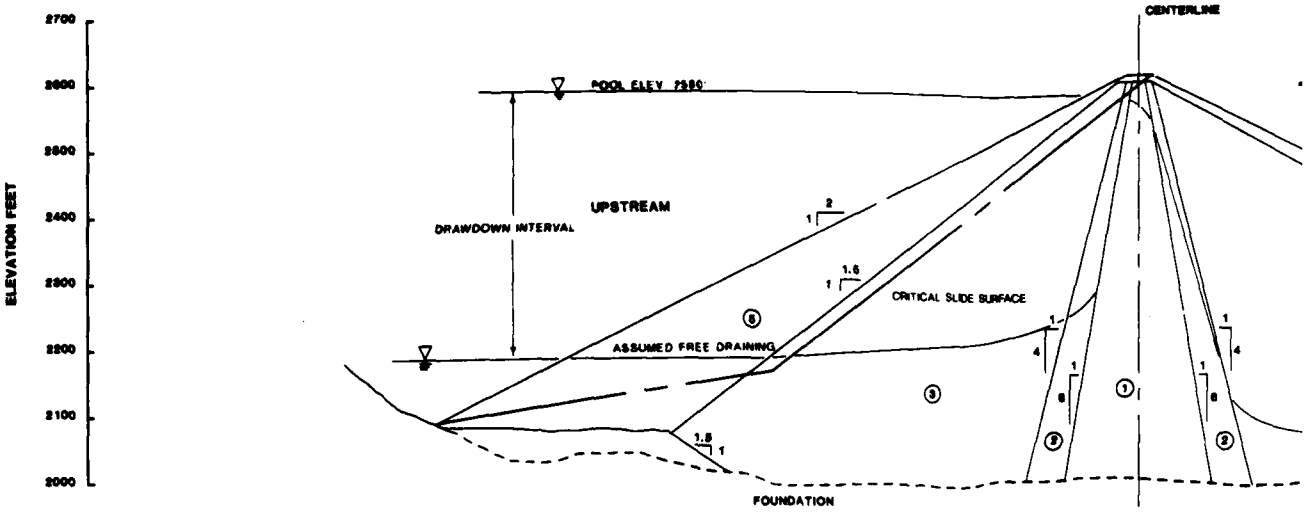
SUBMITTED BY: DATE APPROVED:
CHIEF, ENGINEERING DIVISION

SPEC. NO. DACW09-____-____
DISTRICT FILE NO.

SHEET

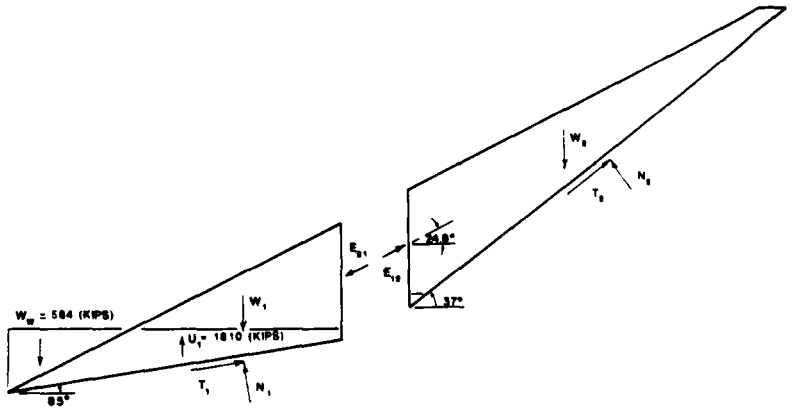
PLATE A-86

SAFETY PAYS



EMBANKMENT SECTION

SCALE 1 INCH : 100 FEET



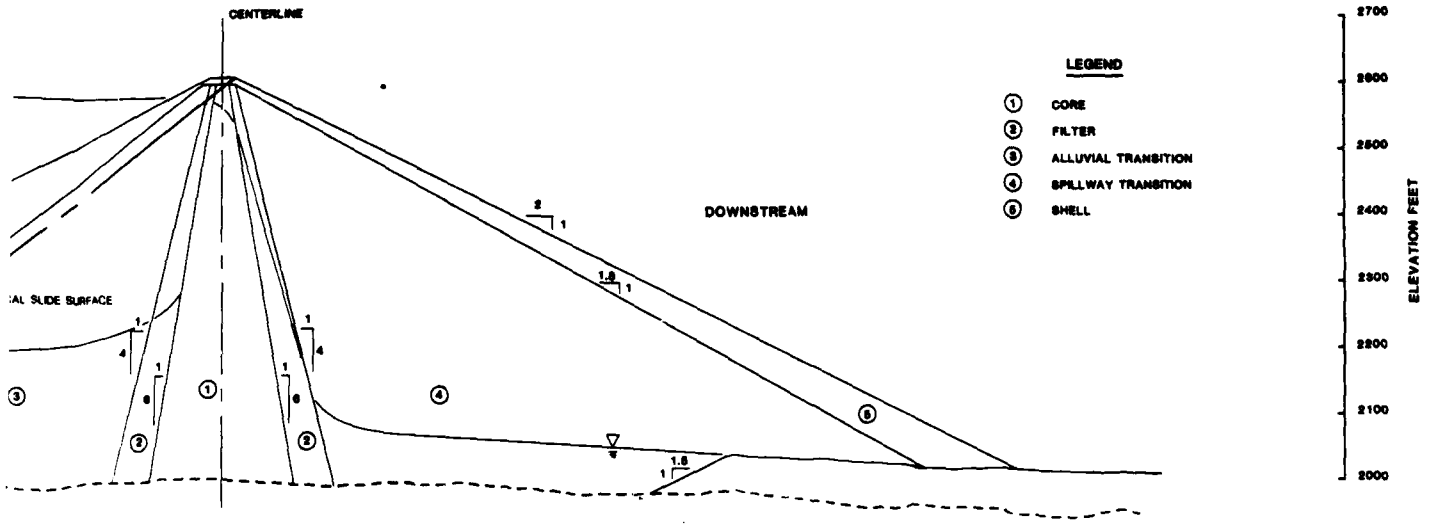
METHOD OF ANALYSIS

SUMMARY OF FORCES

	WEDGE/ BLOCK	WEIGHT (W) (KIPS)	TANGENT (T) (KIPS)	NORMAL (N) (KIPS)	RESULTANT (R) (KIPS)	RESULTANT (R) (DEG)	SIDE FORCE (E) (KIPS)	SIDE FORCE (DEG)
TRIAL ϕ 1	1	6636	2734	5916	6517	74.5	2014	24.8
	2	8339	3206	6954	7657	104.0		
TRIAL ϕ 2	1	6636	2598	5876	6425	73.5	1873	24.8
	2	8339	3085	7142	7768	102.5		

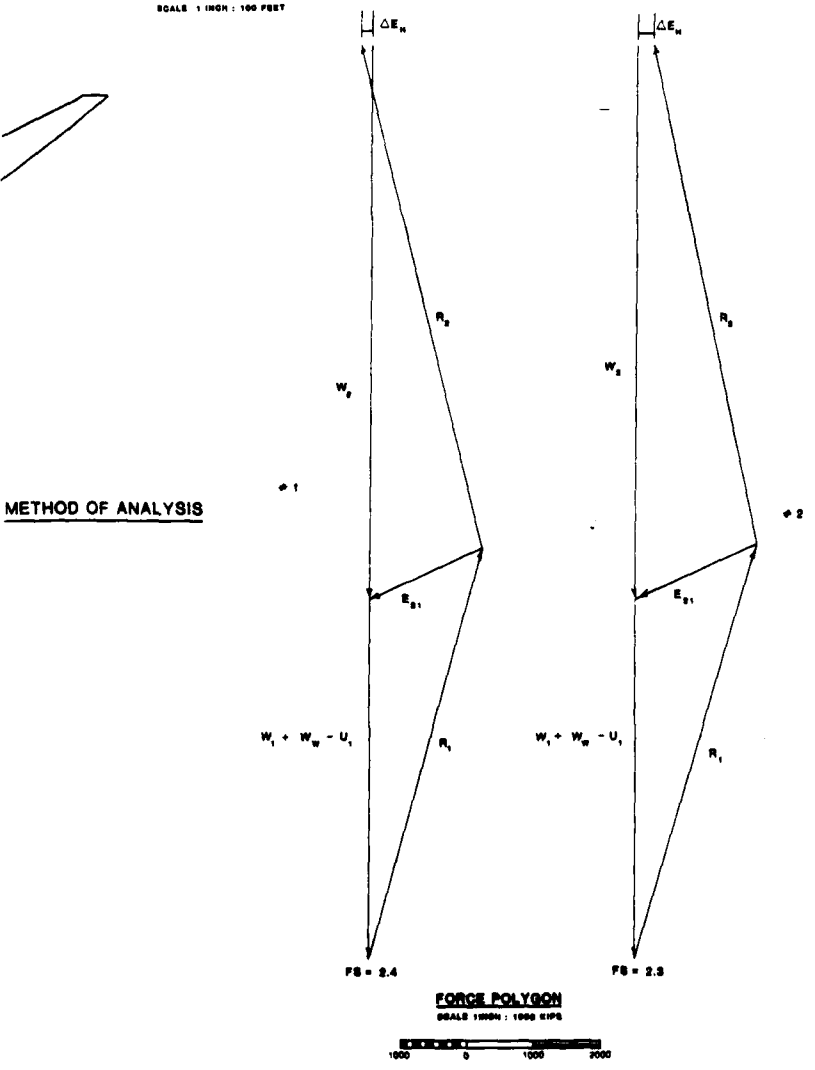
NOTES: SEE NOTES ON PLATE A-76

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING



EMBANKMENT SECTION

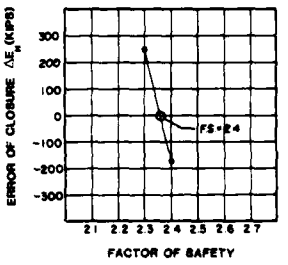
SCALE 1 INCH : 100 FEET



FORCE POLYGON

SCALE 1 INCH : 1000 KIPS

1000 0 1000 2000



METHOD OF ANALYSIS

GENERAL NOTES :

1. TRIAL WEDGES WERE ANALYZED AND CHECKED BY HAND USING THE GRAPHICAL WEDGE METHOD IN ACCORDANCE WITH EM 1110-2-1902.
2. FOUNDATION SHEAR STRENGTH EXCEEDS EMBANKMENT SHEAR STRENGTH.

DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

SYMBOL	DESCRIPTION	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

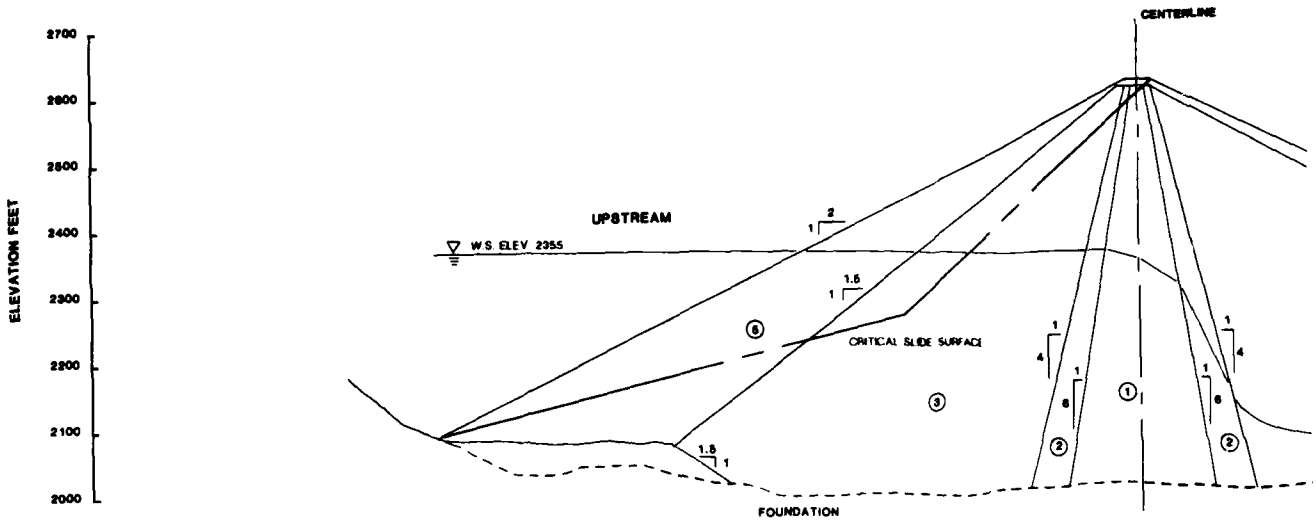
SEVEN OAKS DAM
EMBANKMENT SLOPE STABILITY ANALYSIS
SUDDEN DRAWDOWN

SUBMITTED BY: DATE APPROVED:
CHIEF, ENGINEERING DIVISION

SPEC. NO. DACW09-____-____
DISTRICT FILE NO.

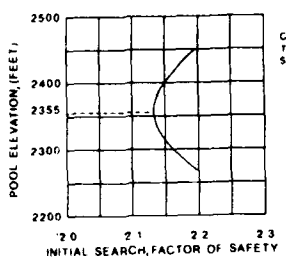
SHEET

PLATE A-87

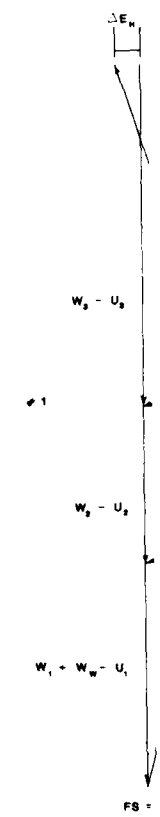


EMBANKMENT SECTION

SCALE 1 INCH = 100 FEET



CRITICAL SLIDE SURFACE USED FOR THE INITIAL SEARCH WAS NOT THE SAME AS THE FINAL SURFACE



METHOD OF ANALYSIS

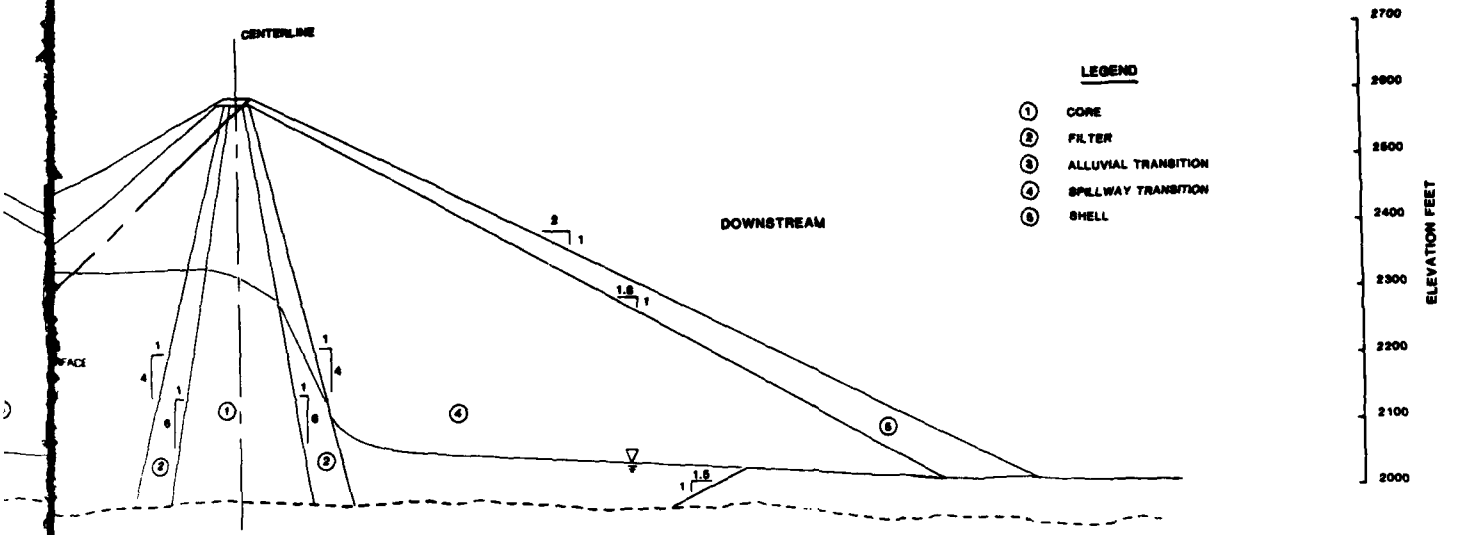
SUMMARY OF FORCES

	WEDGE/ BLOCK	WEIGHT (W) (KIPS)	UPLIFT (U) (KIPS)	TANGENT (T) (KIPS)	NORMAL (N) (KIPS)	RESULTANT (R) (KIPS)	RESULTANT (R) (DEG)	SIDE FORCE (E) (KIPS)	SIDE FORCE (E) (DEG)
TRIAL φ 1	1	5707	7127	1715	3267	3690	76.5	907	15.6
	2	3463	1117	902	2291	2462	82.0	1261	15.6
	3	5471	348	2020	4619	5041	108.5		
TRIAL φ 2	1	5707	7127	2036	3276	3857	72.0	1228	15.6
	2	3463	1117	1068	2290	2527	79.0	1021	15.6
	3	5471	348	2214	4274	4814	104.5		

NOTES: SEE NOTES ON PLATE A-76

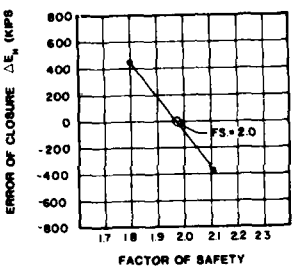
ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

AY ALUE ENGINEERING PAYS

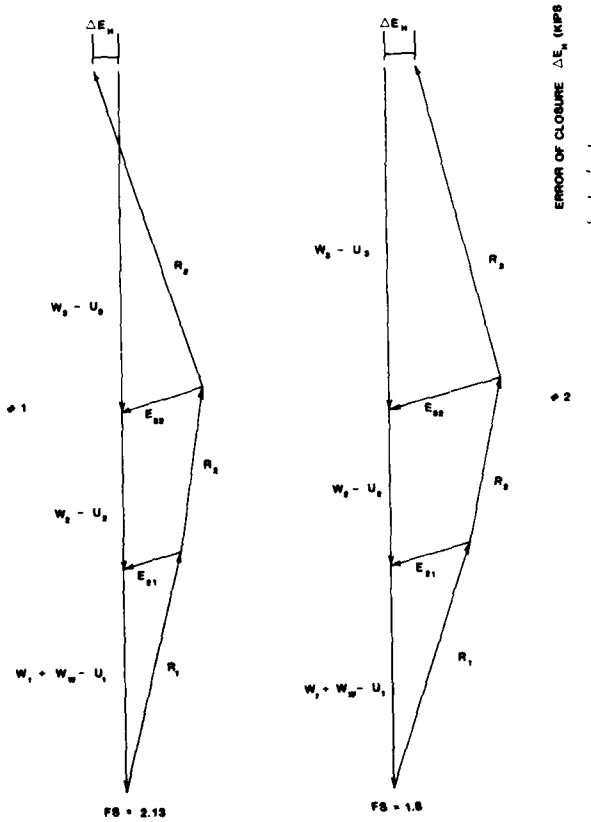


EMBANKMENT SECTION

SCALE 1 INCH = 100 FEET



METHOD OF ANALYSIS



FORCE POLYGON

SCALE 1 INCH = 1000 KIPS

1000 0 1000 2000

GENERAL NOTES:

1. TRIAL WEDGES WERE ANALYZED AND CHECKED BY HAND USING THE GRAPHICAL WEDGE METHOD IN ACCORDANCE WITH EM 1110-2-1902.
2. FOUNDATION SHEAR STRENGTH EXCEEDS EMBANKMENT SHEAR STRENGTH.

DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

SYMBOL	DESCRIPTION	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM
EMBANKMENT SLOPE STABILITY ANALYSIS
PARTIAL POOL

SUBMITTED BY: DATE APPROVED:

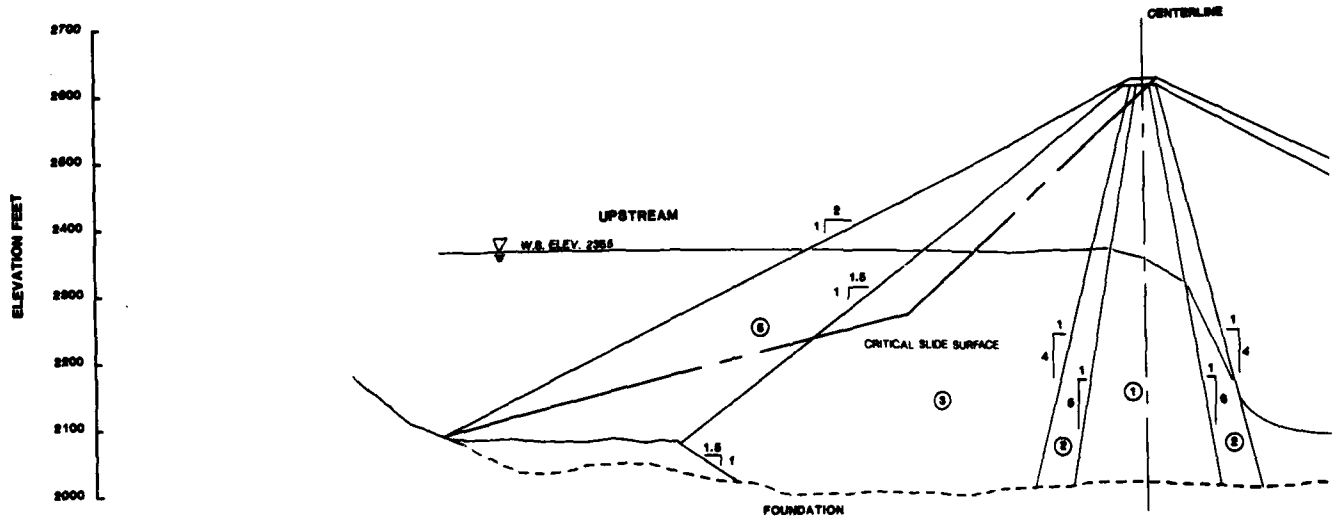
SPEC. NO. DACW09- - -

DISTRICT FILE NO.

SHEET

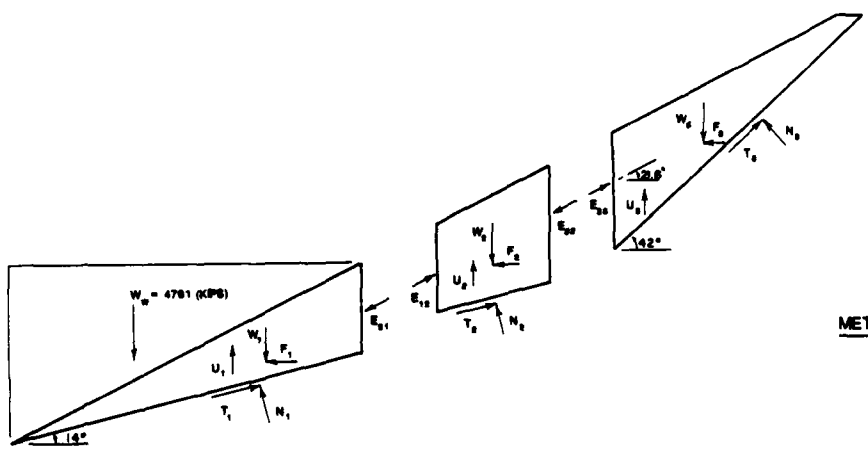
PLATE A-88

SAFETY PAYS

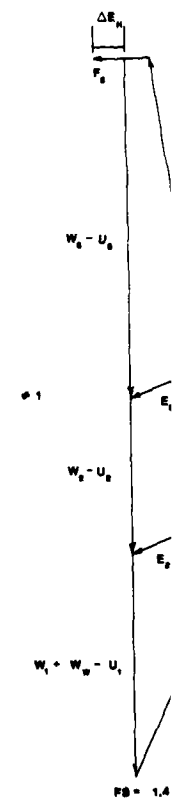


EMBANKMENT SECTION

SCALE 1 INCH = 100 FEET



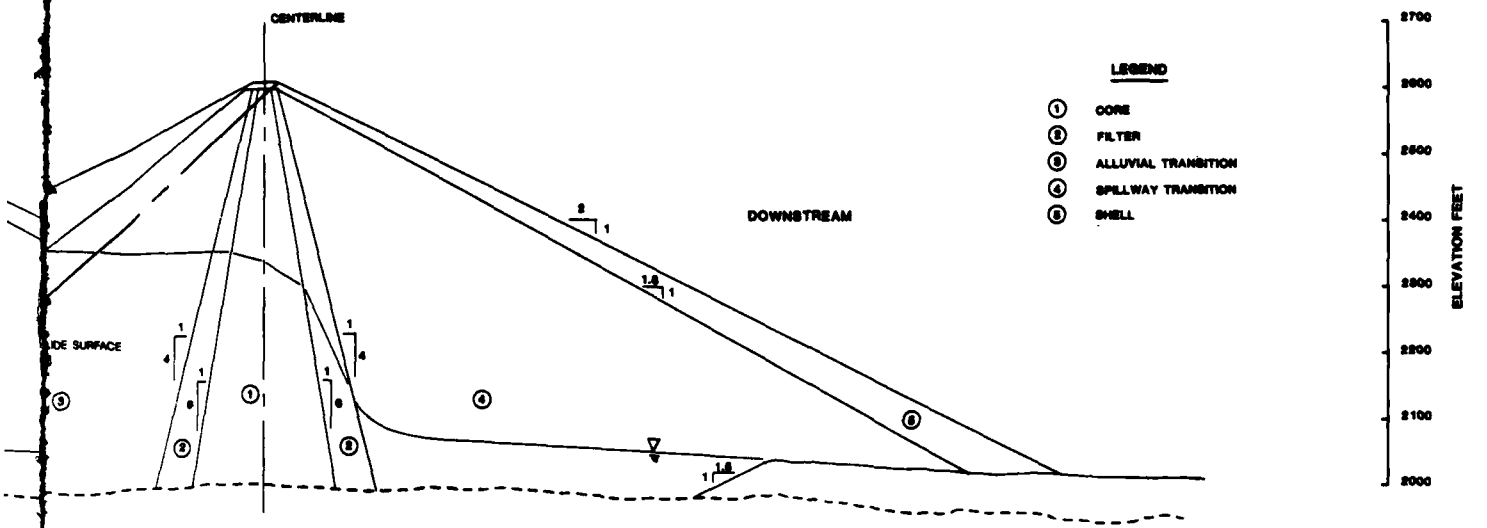
METHOD OF ANALYSIS



SUMMARY OF FORCES

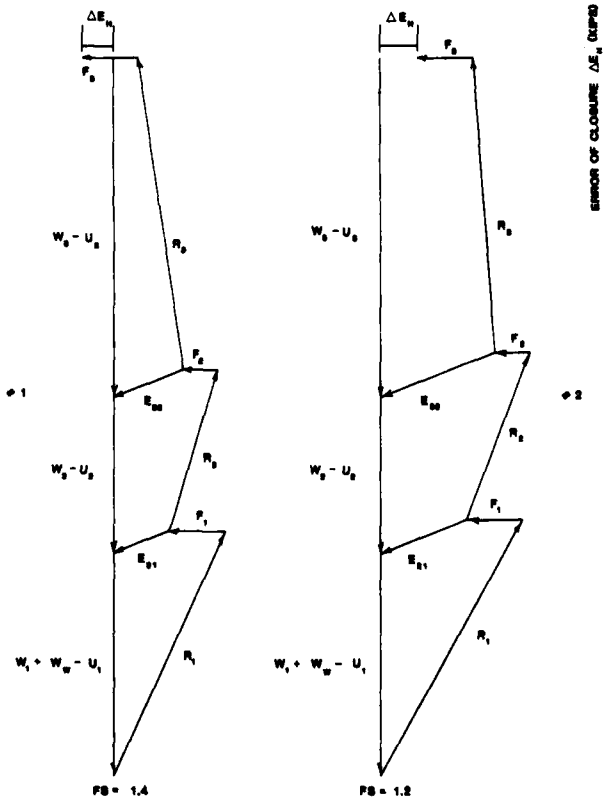
	WEDGE/ BLOCK	WEIGHT (W) (KIPS)	UPLIFT (U) (KIPS)	SEISMIC (F) (KIPS)	TANGENT (T) (KIPS)	NORMAL (N) (KIPS)	RESULTANT (R) (KIPS)	(DEG)	SIDE FORCE (E) (KIPS)	(DEG)
TRIAL φ 1	1	5707	7127	856	2517	3150	4032	66.0	886	21.6
	2	3463	1117	520	1306	2183	2545	73.5	1128	21.6
	3	8471	348	821	2838	3961	4759	98.2		
TRIAL φ 2	1	5707	7127	856	2986	3214	4393	60.5	1369	21.6
	2	3463	1117	520	1549	2215	2703	69.0	1851	21.6
	3	8471	348	821	2732	3516	4453	94.0		

NOTES: SEE NOTES ON PLATE A-76



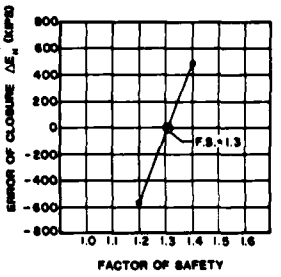
EMBANKMENT SECTION

SCALE 1 INCH : 100 FEET



FORCE POLYGON

SCALE 1 INCH = 1000 KIPS



METHOD OF ANALYSIS

GENERAL NOTES :

1. TRIAL WEDGES WERE ANALYZED AND CHECKED BY HAND USING THE GRAPHICAL WEDGE METHOD IN ACCORDANCE WITH EM 1110-2-1902.
2. FOUNDATION SHEAR STRENGTH EXCEEDS EMBANKMENT SHEAR STRENGTH.
3. SEISMIC COEFFICIENT OF 0.15 FOR ZONE 4.

DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

SYMBOL	DESCRIPTION	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

EMBANKMENT SLOPE STABILITY ANALYSIS
PARTIAL POOL WITH EARTHQUAKE

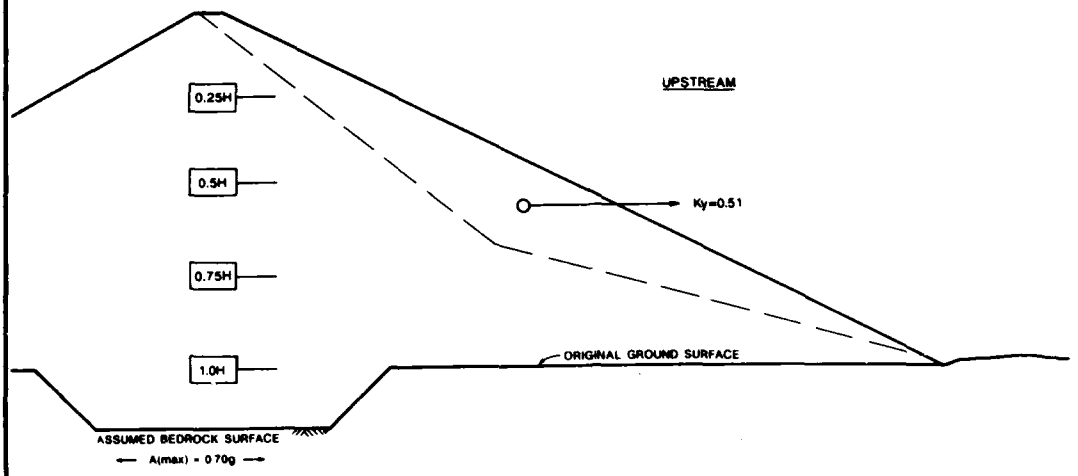
SUBMITTED BY: DATE APPROVED

SPEC. NO. DACW09-__-__-__

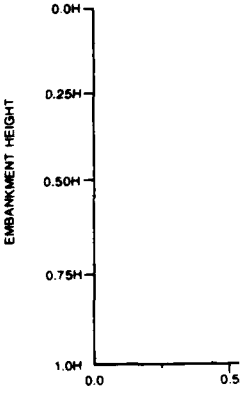
DISTRICT FILE NO.

SHEET

PLATE A-09

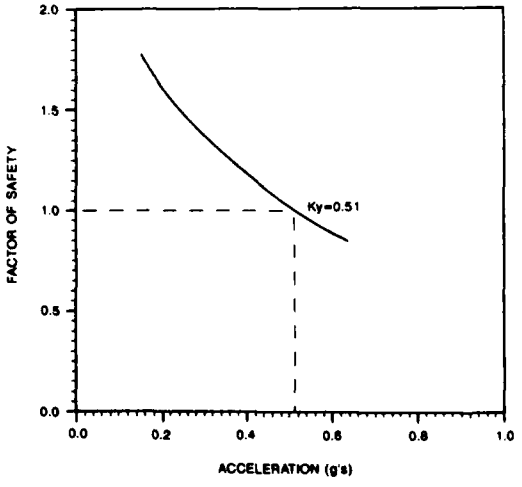


ASSUMED EMBANKMENT SECTION
(N T S)



ESTIMATED

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING



DETERMINATION OF YIELD ACCELERATION

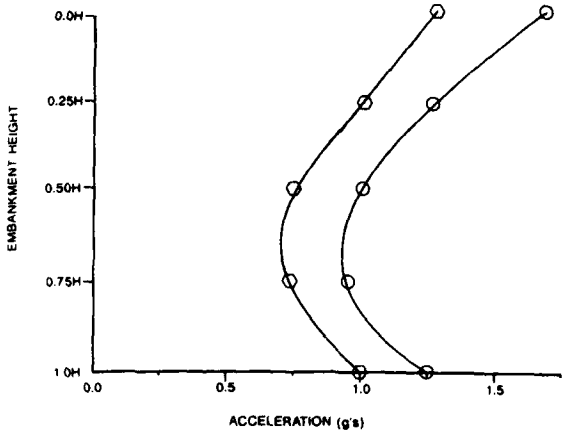
NOTES:

1 Ky DENOTES YIELD ACCELERATION

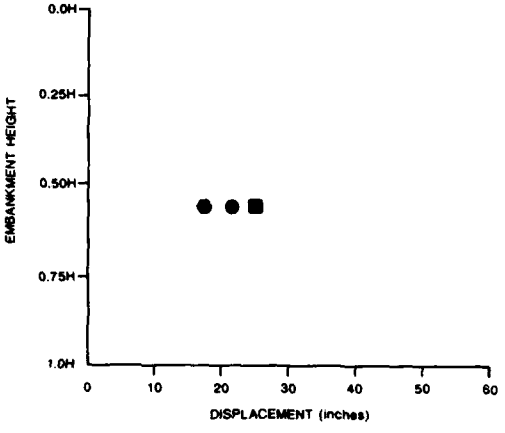
2 DENOTES ACCELERATION (MA
BOLT'S MAGNITUDE 8- EARTHQ
SEED-IDRISS MAGNITUDE 8- EA

3 DENOTES ESTIMATED DISPLACE
NEWMARK/SARMA/AMBRAY
BOLT'S MAGNITUDE 8- EARTHQ
SEED-IDRISS MAGNITUDE 8- EA
MAKDISI-SEED METHOD
(AVG. DISPLACEMENT FOR BOLT'S

4 THE ACCELERATION VALUES AT THE ORIG
OBTAINED FROM THE 'SHAKE' COMPUTER
THROUGH THE FOUNDATION ALLUVIUM



ESTIMATED ACCELERATION PROFILE



ESTIMATED SLOPE DISPLACEMENT

NOTES:

1 Ky DENOTES YIELD ACCELERATION

2 DENOTES ACCELERATION (MAXIMUM CREDIBLE EVENT A(max) = 0.70g)
- BOLT'S MAGNITUDE 8+ EARTHQUAKE
- SEED-IDRISS MAGNITUDE 8+ EARTHQUAKE

3 DENOTES ESTIMATED DISPLACEMENTS
- NEWMARK/SARMA/AMBRAYSEYS METHOD
  - BOLT'S MAGNITUDE 8+ EARTHQUAKE
  - SEED-IDRISS MAGNITUDE 8+ EARTHQUAKE
- MAKDISI-SEED METHOD
  (AVG. DISPLACEMENT FOR BOLT'S 8+ EARTHQUAKE)

4 THE ACCELERATION VALUES AT THE ORIGINAL GROUND SURFACE ARE FREE FIELD VALUES OBTAINED FROM THE 'SHAKE' COMPUTER PROGRAM BY PROPAGATING BEDROCK MOTIONS THROUGH THE FOUNDATION ALLUVIUM.

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

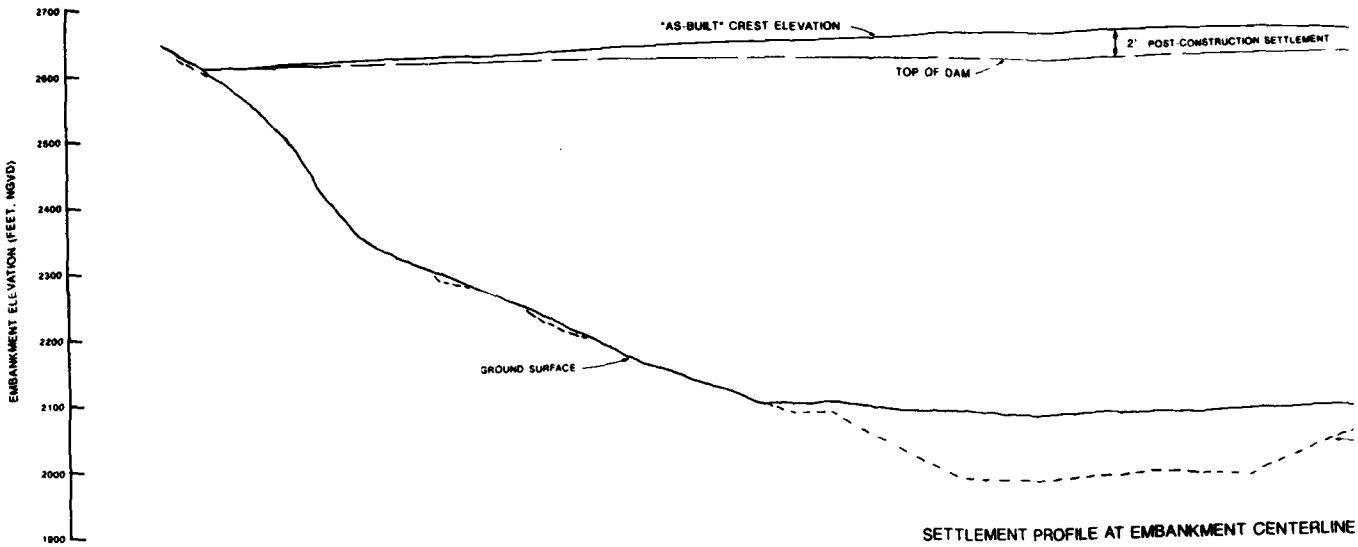
PRELIMINARY ANALYSIS OF PERMANENT EMBANKMENT DEFORMATION

SUBMITTED BY: DATE
APPROVED:
CHIEF, DIVISION

SPEC. NO. DACW09-____-____
DISTRICT FILE NO.

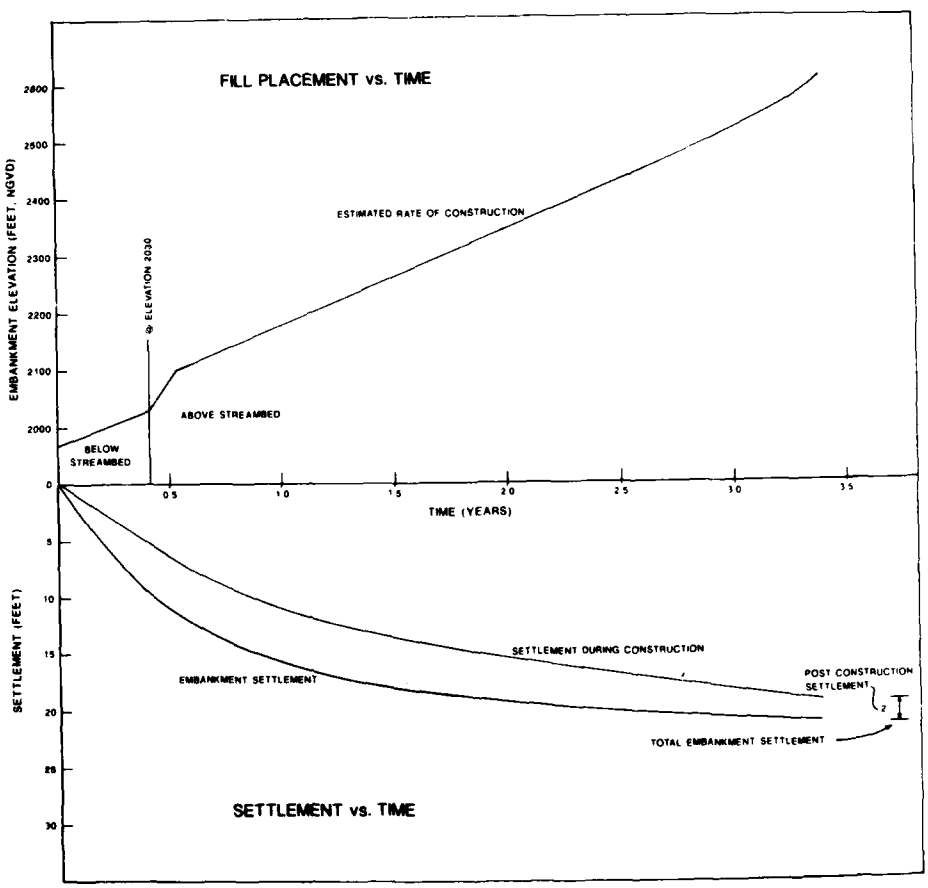
SHEET

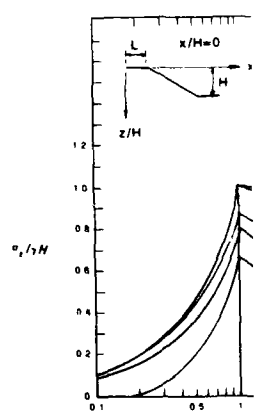
PLATE A-90



SETTLEMENT PROFILE AT EMBANKMENT CENTERLINE

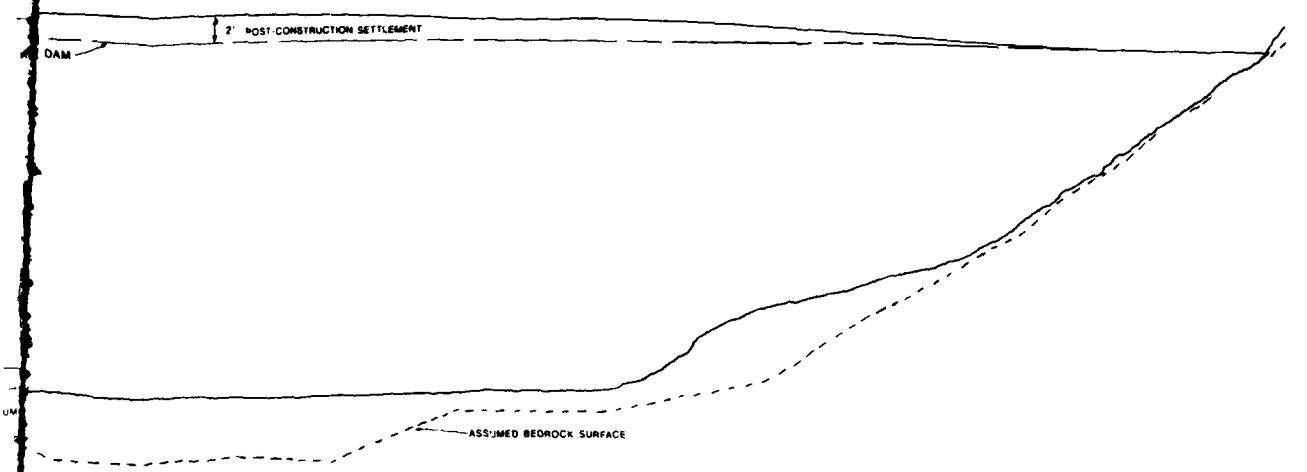
ENVIRONMENTAL
ENHANCEMENT
THRU ENGINEERING





FILL PLACEMENT AND SETTLEMENT RATES

INFLI



ETTLEMENT PROFILE AT EMBANKMENT CENTERLINE

LEGEND:

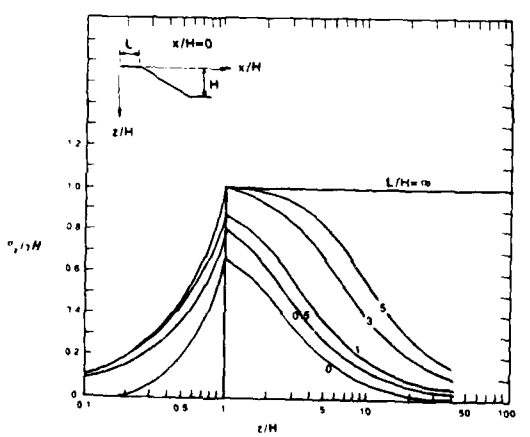
——— GROUND SURFACE

– – – ASSUMED BEDROCK SURFACE

—·—·— EMBANKMENT CREST ELEVATION

NOTES:

1. INFLUENCE DIAGRAM FOR VERTICAL NORMAL STRESS ALONG SELECTED SECTIONS DUE TO ELASTIC EMBANKMENT $\nu = 0.3$, $\alpha = 30°$ (AFTER PERLOFF et al 1967)



INFLUENCE DIAGRAM

DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

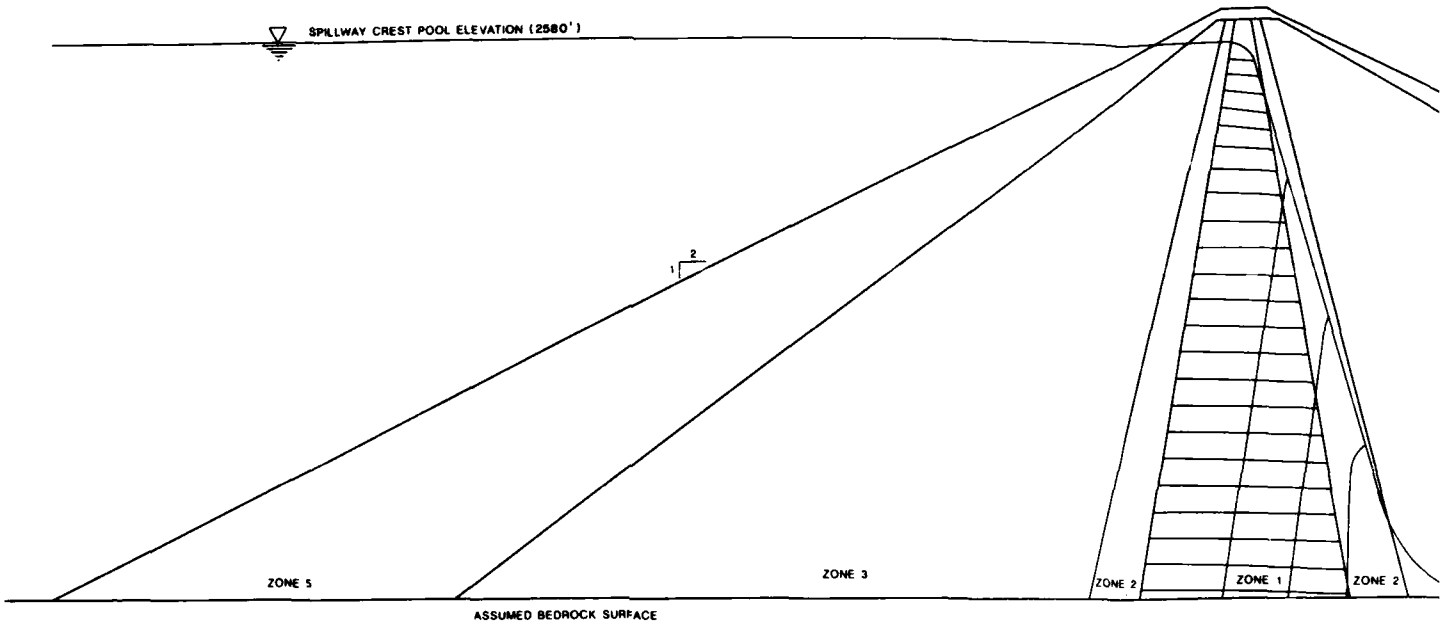
SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

EMBANKMENT SETTLEMENT ANALYSIS

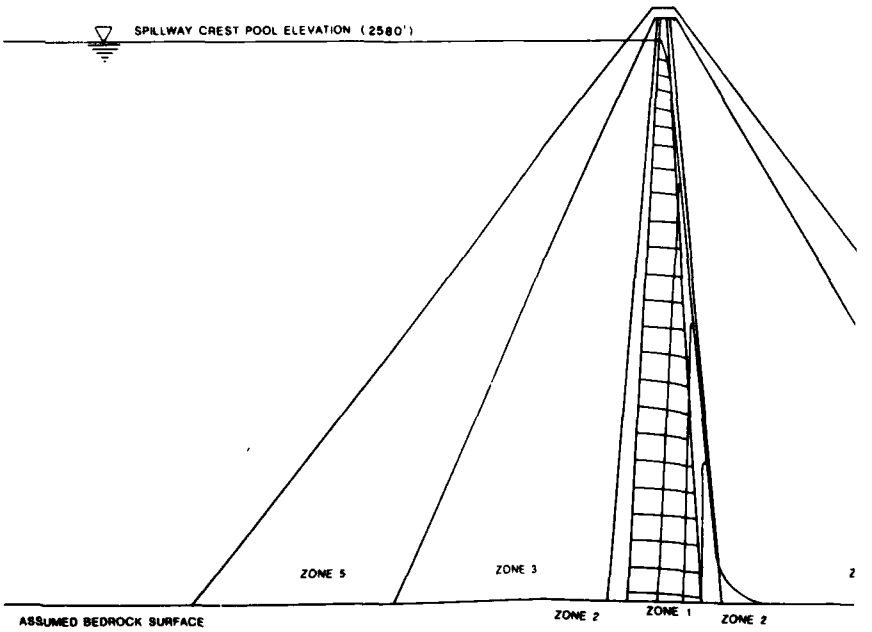
DESIGNED BY:
DRAWN BY:
CHECKED BY:
SUBMITTED BY:
DATE APPROVED:
SPEC. NO. DACW09-____-____
DISTRICT FILE NO
SHEET

PLATE A-91



TRUE SECTION

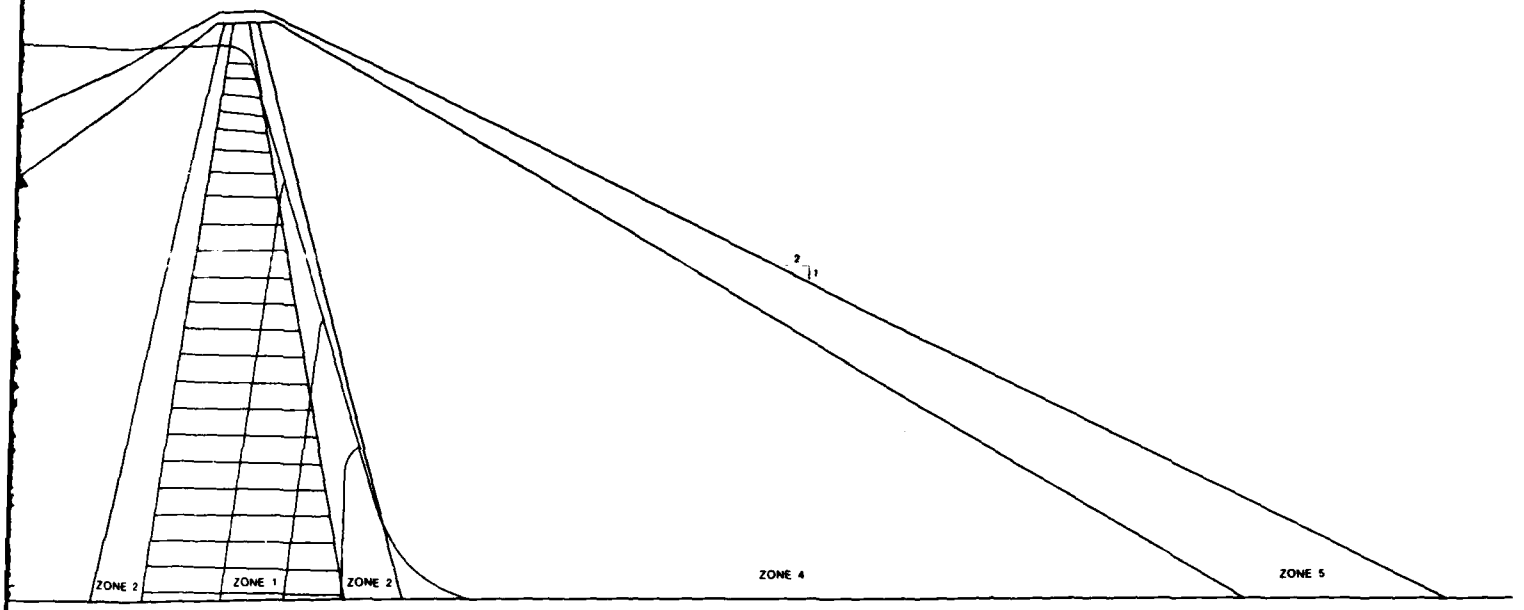
SCALE 50 0 50 100 150 FEET



TRANSFORMED SECTION

SCALE 50 0 50 100 150 FEET

ENVIRONMENTAL
ENHANCEMENT
THRU ENGINEERING



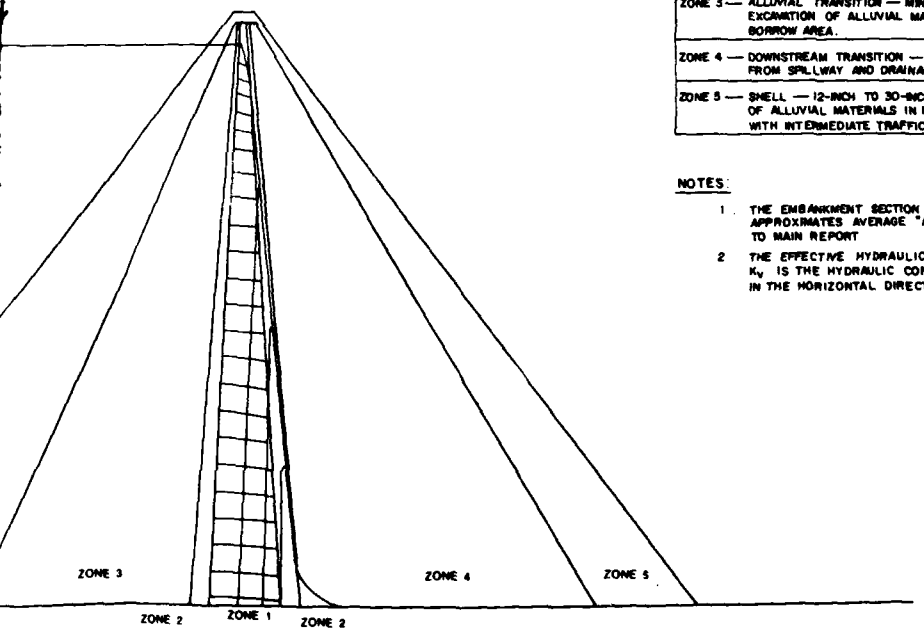
TRUE SECTION

SCALE 50 0 50 100 150 FEET

DESCRIPTION OF MATERIAL	EFFECTIVE HYDRAULIC CONDUCTIVITY, K' (FT/DAY)
ZONE 1 — IMPERVIOUS — CLAYEY SANDS AND SILTY SANDS FROM IMPERVIOUS BORROW AREA.	0.10
ZONE 2 — FILTER — MINUS 3-INCH SAND AND GRAVEL PROCESSED FROM EXCAVATION OF ALLUVIAL MATERIALS IN FOUNDATION AND DOWNSTREAM PERVIOUS BORROW AREA.	30-70
ZONE 3 — ALLUVIAL TRANSITION — MINUS 12-INCH SAND, GRAVEL AND COBBLES PROCESSED FROM EXCAVATION OF ALLUVIAL MATERIALS IN FOUNDATION AND DOWNSTREAM PERVIOUS BORROW AREA.	120-240
ZONE 4 — DOWNSTREAM TRANSITION — SAND SIZE TO APPROXIMATELY 9-INCH ROCK FRAGMENTS FROM SPILLWAY AND DRAINAGE CHANNEL EXCAVATION.	180-300
ZONE 5 — SHELL — 12-INCH TO 30-INCH COBBLES AND BOULDERS PROCESSED FROM EXCAVATION OF ALLUVIAL MATERIALS IN FOUNDATION AND DOWNSTREAM PERVIOUS BORROW AREA, WITH INTERMEDIATE TRAFFIC LAYERS OF ZONE 3 MATERIAL.	3,000-5,000

NOTES:

1. THE EMBANKMENT SECTION UTILIZED IN THE SEEPAGE ANALYSIS IS AN HYPOTHETICAL SECTION WHICH APPROXIMATES AVERAGE "AS-CONSTRUCTED" CONDITIONS. FOR ACTUAL EMBANKMENT SECTION, REFER TO MAIN REPORT
2. THE EFFECTIVE HYDRAULIC CONDUCTIVITY (PERMEABILITY) OF A SOIL IS DEFINED AS $K' = \sqrt{K_V \cdot K_H}$, WHERE K_V IS THE HYDRAULIC CONDUCTIVITY IN THE VERTICAL DIRECTION AND K_H, THE HYDRAULIC CONDUCTIVITY IN THE HORIZONTAL DIRECTION



TRANSFORMED SECTION

SCALE 50 0 50 100 150 FEET

UNITY FLOW QUANTITY

$q = K' \frac{\Delta h}{L_e} h$

$q = (0.1 \text{ fpd})\left(\frac{23.25}{5}\right)(500 \text{ FT}) = 291 \frac{\text{CFD}}{\text{FT}}$

DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM
EMBANKMENT SEEPAGE ANALYSIS
CASE I - SPILLWAY CREST ELEVATION
(INTACT CORE)

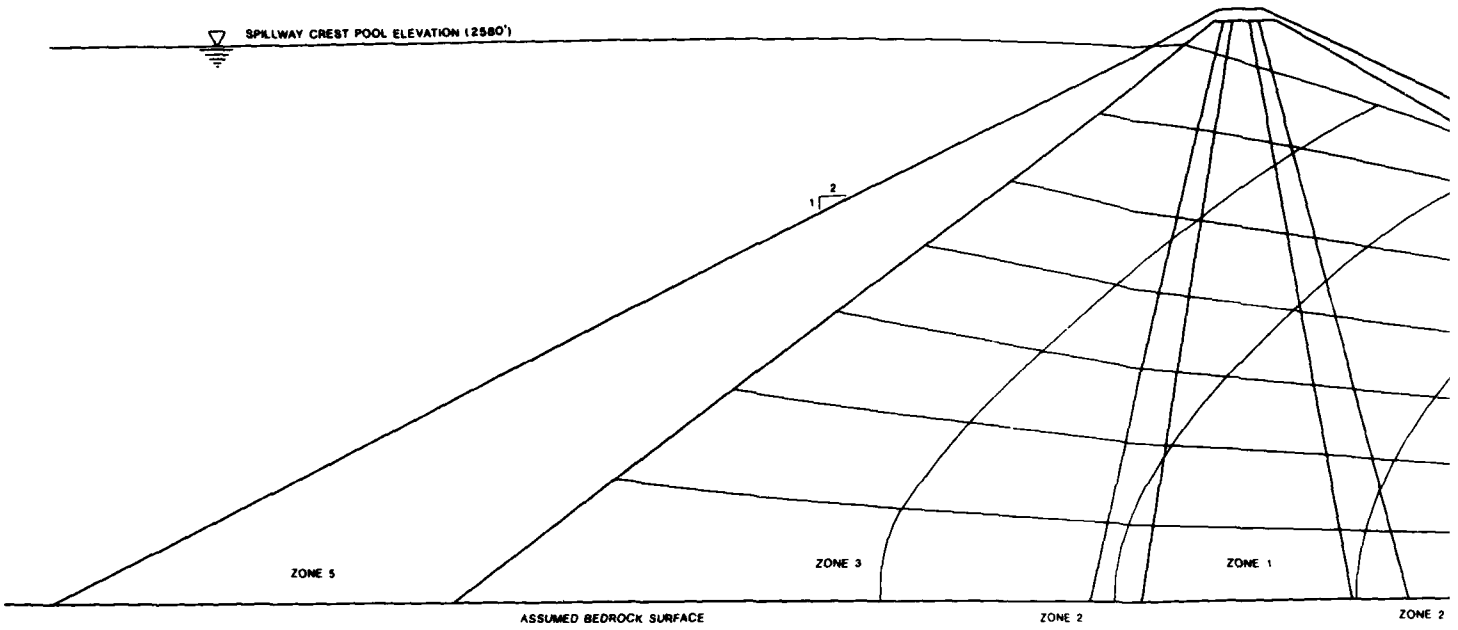
SUBMITTED BY: DATE APPROVED:

SPEC. NO. DACW09-___-___-___

DISTRICT FILE NO.

SHEET

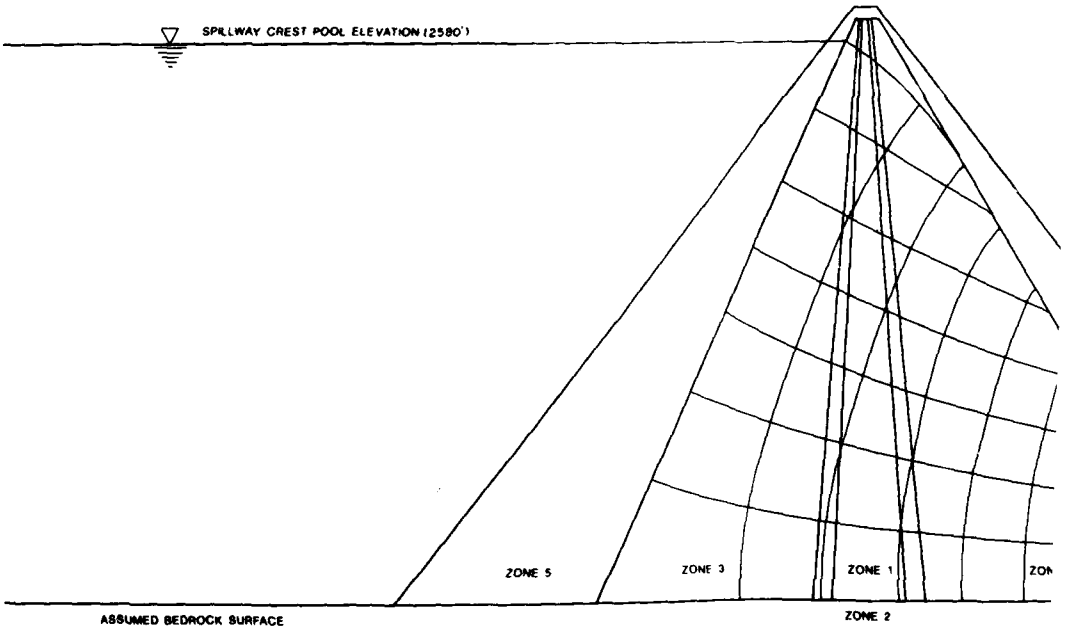
PLATE A-92



TRUE SECTION

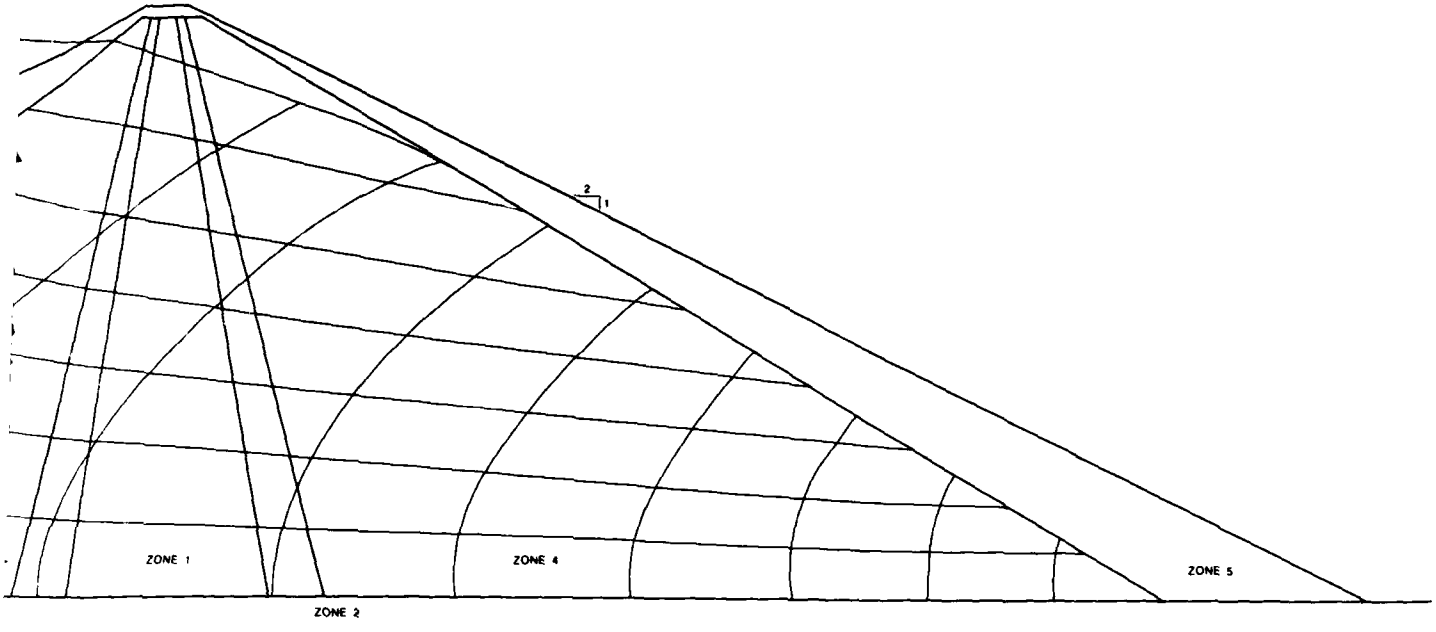
SCALE 50 0 50 100 150 FEET

ENVIRONMENTAL
ENHANCEMENT
THRU ENGINEERING



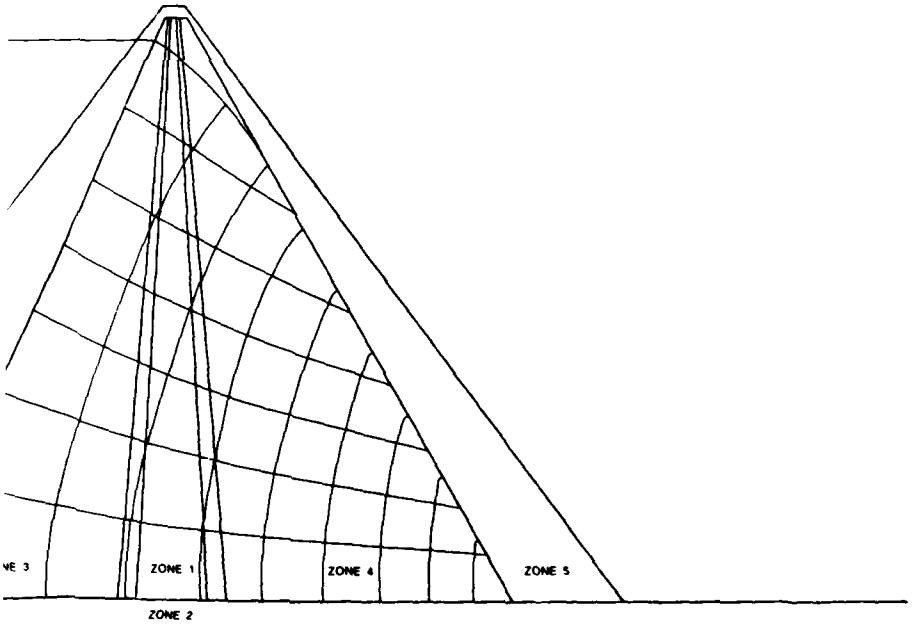
TRANSFORMED SECTION

SCALE 50 0 50 100 150 FEET



TRUE SECTION

SCALE 50 0 50 100 150 FEET



TRANSFORMED SECTION

SCALE 50 0 50 100 150 FEET

UNIT FLOW QUANTITY

$q = K' \frac{n_f}{n_d} h$

$q = (.70\,fpd\,K^{7/8}H\,500\,ft) = 27{,}222\ \frac{cfd}{ft.}$

NOTES:

1. REFER TO PLATE A-92 FOR GENERAL NOTES AND DESCRIPTION OF MATERIALS.

DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

SYMBOL	DESCRIPTION	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
TRACED BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

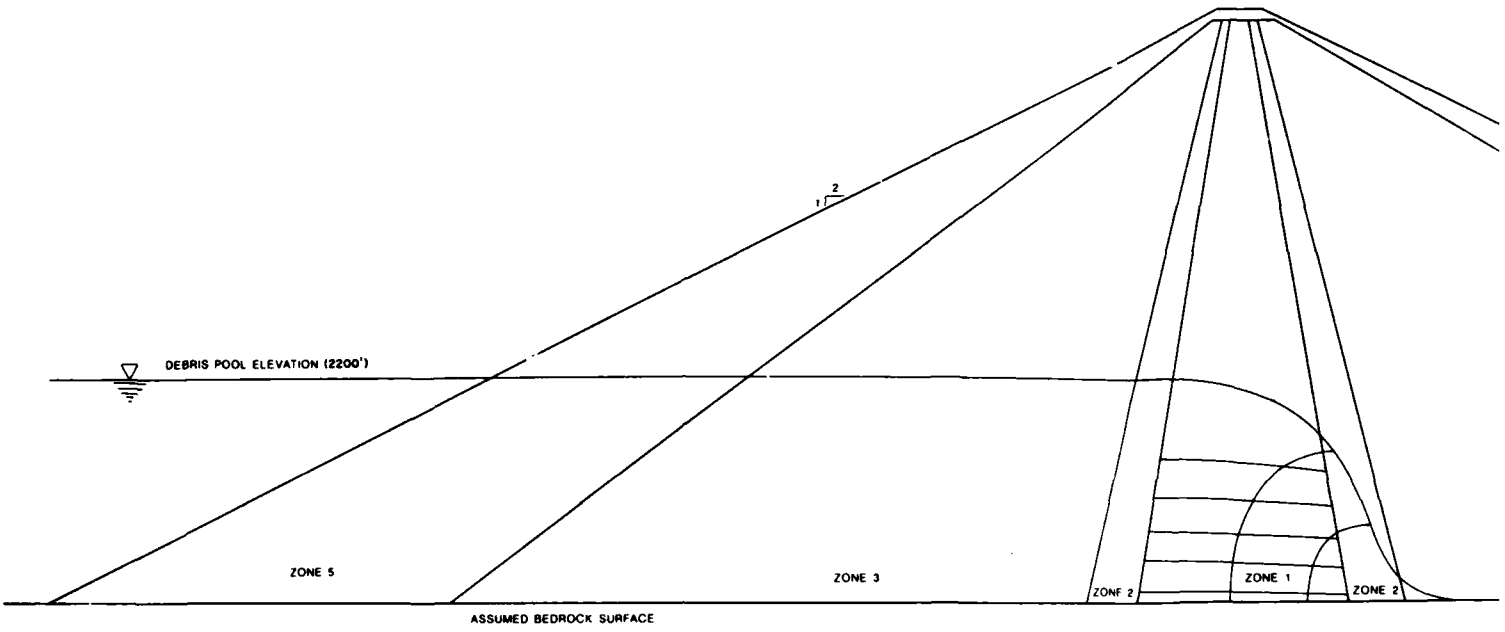
SEVEN OAKS DAM
EMBANKMENT SEEPAGE ANALYSIS
CASE II - SPILLWAY CREST ELEVATION
(CRACKED CORE)

SUBMITTED BY: DATE APPROVED:
CHIEF, ENGINEERING DIVISION

SPEC. NO. DACW09-___-___
DISTRICT FILE NO.

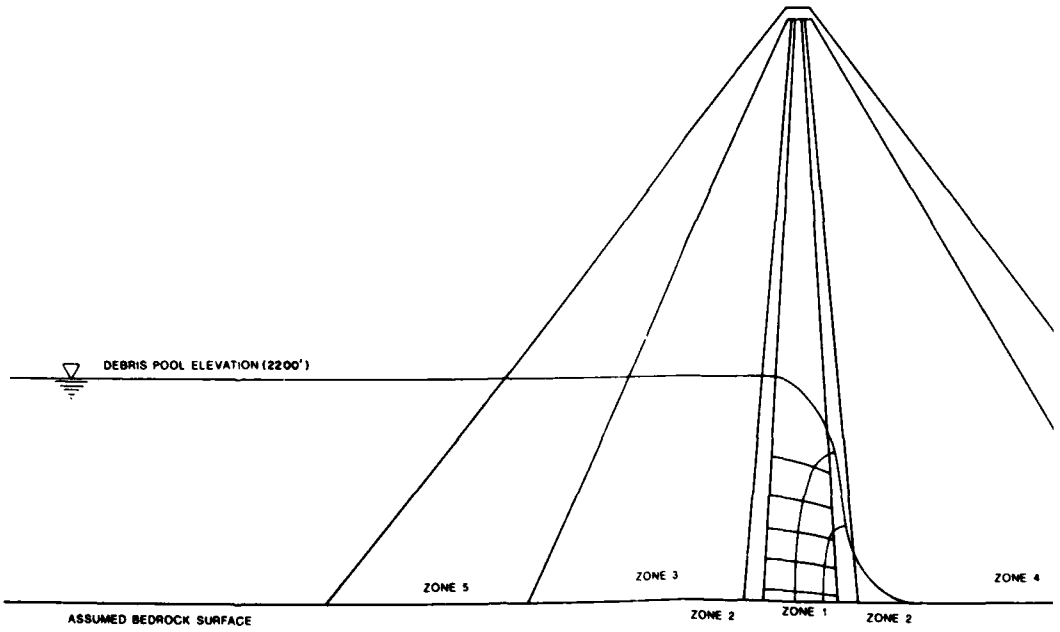
SHEET

PLATE A-93



TRUE SECTION

SCALE 50 0 50 100 150 FEET

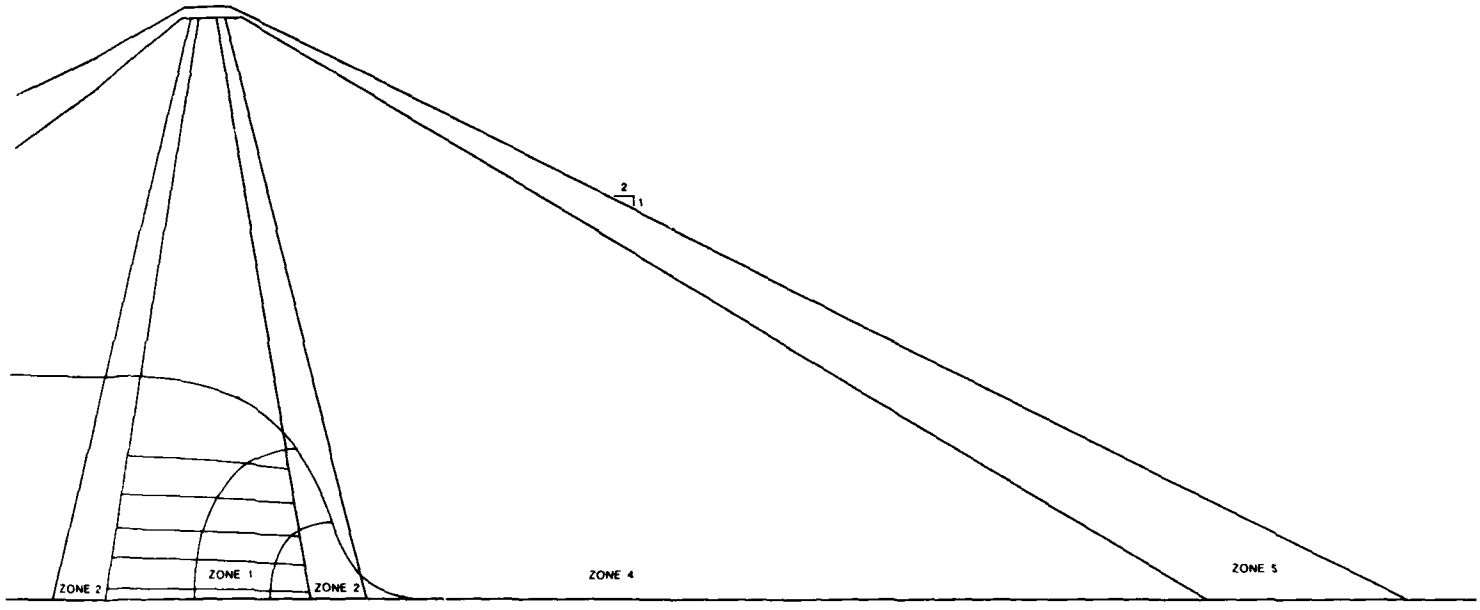


TRANSFORMED SECTION

SCALE 50 0 50 100 150 FEET

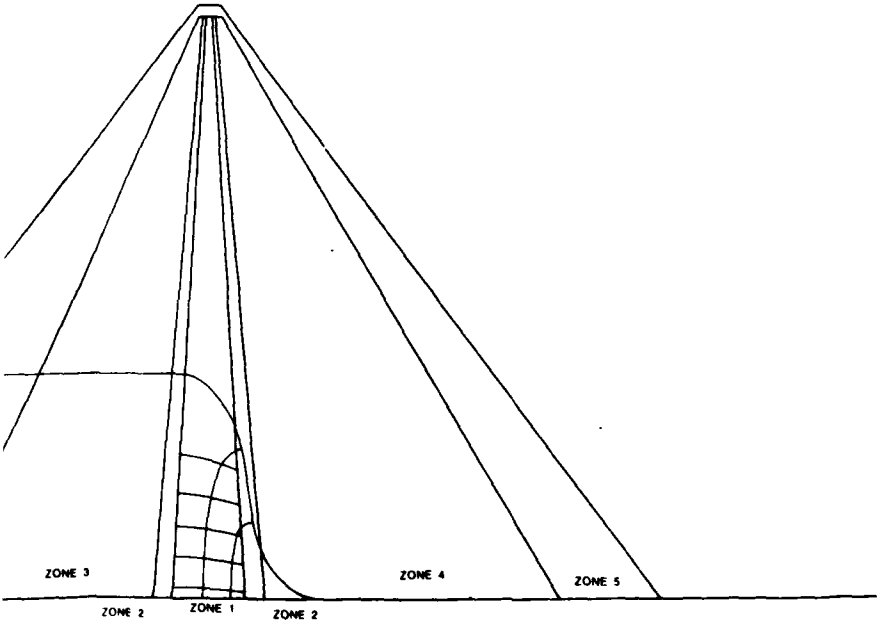
ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

ALUE ENGINEERING PAYS



TRUE SECTION

SCALE 50 0 50 100 150 FEET



TRANSFORMED SECTION

SCALE 50 0 50 100 150 FEET

UNIT FLOW QUANTITY

$q = K' \frac{n_f}{n_d} h$

$q = (0.1 fpd)(\frac{5.25}{3})(200 FT.) = 35 \frac{cfd}{ft}$

NOTES.

1. REFER TO PLATE A-92 FOR GENERAL NOTES AND DESCRIPTION OF MATERIALS.

DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY.
DRAWN BY.
CHECKED BY.

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

EMBANKMENT SEEPAGE ANALYSIS
CASE III - DEBRIS POOL ELEVATION

SUBMITTED BY: DATE APPROVED:

CHIEF, BRANCH

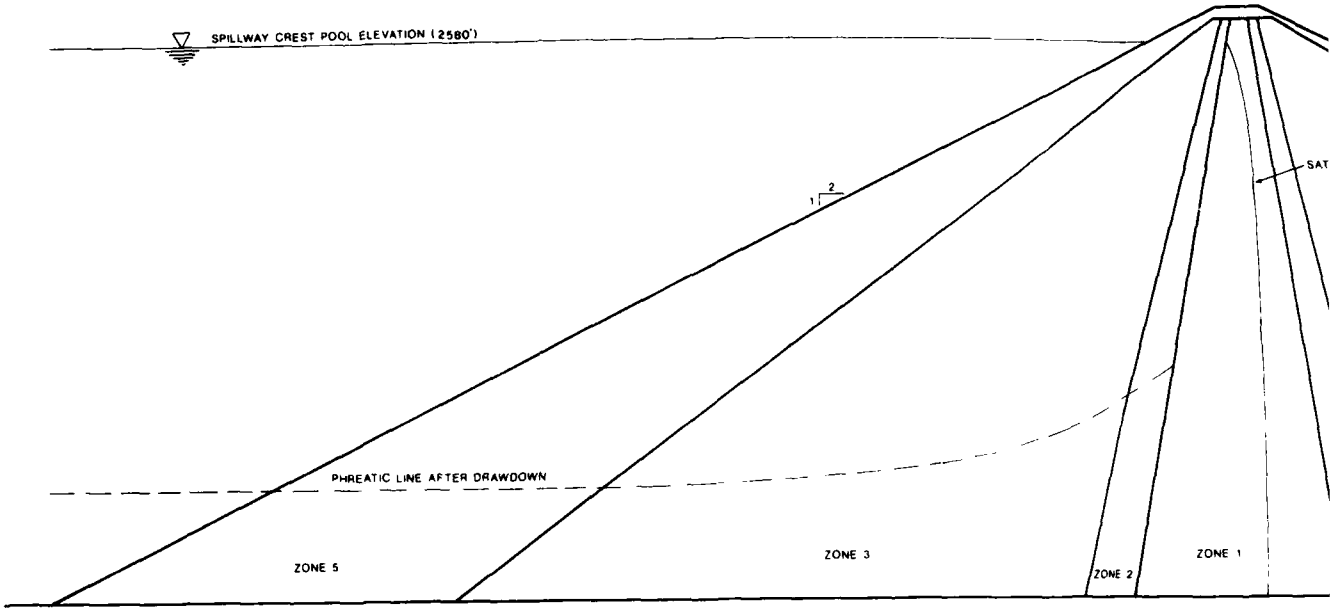
SPEC. NO. DACW09- - -

DISTRICT FILE NO.

SHEET

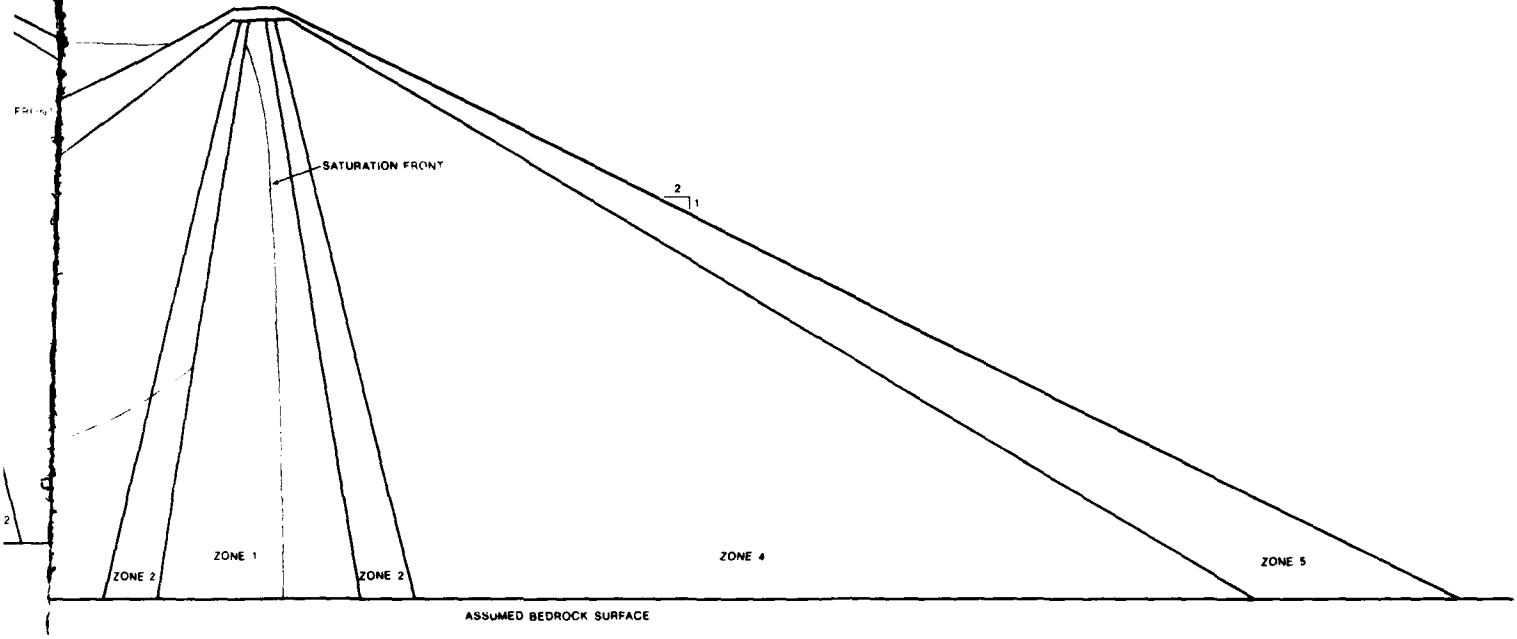
SAFETY PAYS

PLATE A-94



ENVIRONMENTAL
ENHANCEMENT
THRU ENGINEERING

ALUE ENGINEERING PAYS



NOTES:

1. REFER TO PLATE A-92 FOR GENERAL NOTES AND DESCRIPTION OF MATERIALS.

DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

EMBANKMENT SEEPAGE ANALYSIS
TRANSIENT SEEPAGE LIMITS

SUBMITTED BY: _____ DATE APPROVED: _____

SPEC. NO. DACW09-____-B-____

DISTRICT FILE NO.

SHEET

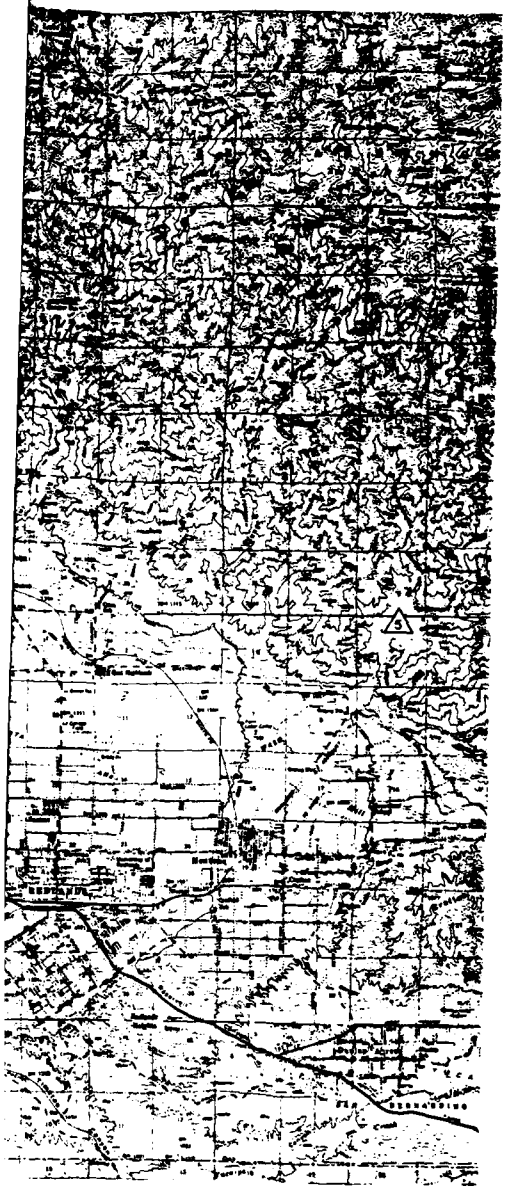
SAFETY PAYS

PLATE A-95



LOCATION MAP

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING



AGGREGATE SOURCES EXAMINED

LYTLE CREEK SOURCES
(SEE LOCATION MAP)

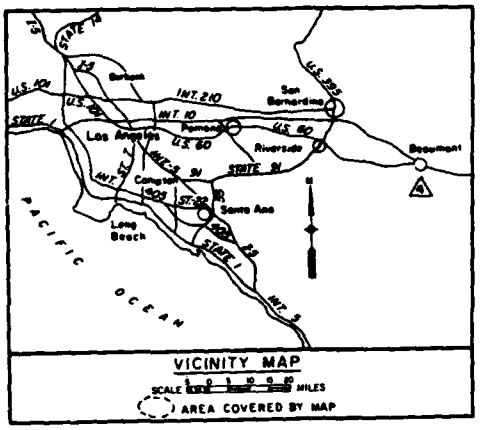
1. OBL ROCK COMPANY
2. 4TH STREET CRUSHER

SANTA ANA RIVER
(SEE LOCATION MAP)

3. LIVINGSTON GRAHAM

SAN GORGONIO RIVER
(SEE VICINITY MAP)

4. BEAUMONT CONCRETE COMPANY
5. ON SITE SOURCE



VICINITY MAP

SCALE 0 10 0 10 20 30 MILES

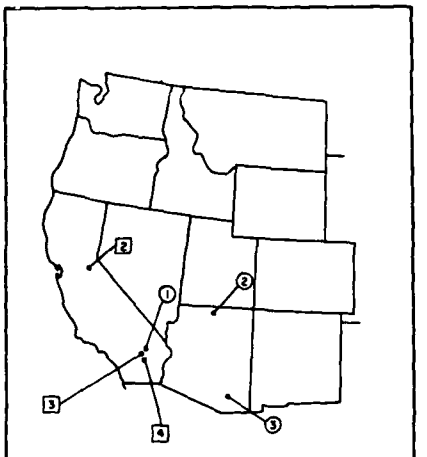
AREA COVERED BY MAP

CEMENT SOURCES

1. CALIFORNIA PORTLAND CEMENT COMPANY (CALMAT)
   COLTON, CALIFORNIA
2. CALAVERAS CEMENT COMPANY
   SAN ANDREAS, CALIFORNIA
3. CALMAT
   MOJAVE, CALIFORNIA
4. RIVERSIDE PORTLAND CEMENT COMPANY
   ORO GRANDE, CALIFORNIA

POZZOLAN SOURCES

1. WESTERN ASH CO. MOJAVE PLANT
   MOJAVE, CALIFORNIA
2. WESTERN ASH CO. NAVAJO PLANT
   PAGE, ARIZONA
3. WESTERN ASH CO. APACHE PLANT
   COCHISE, ARIZONA



GENERAL MAP

NO SCALE

DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

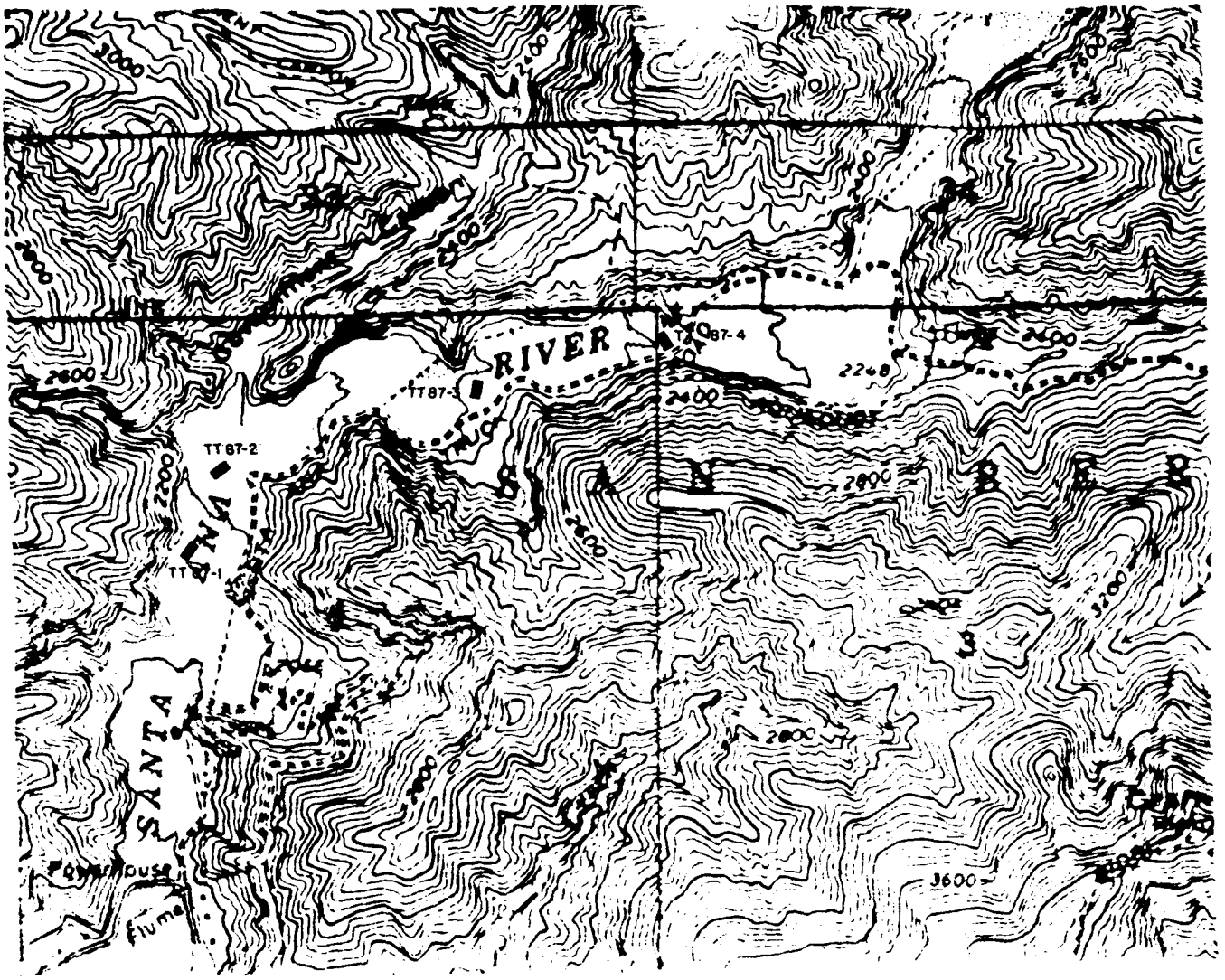
DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

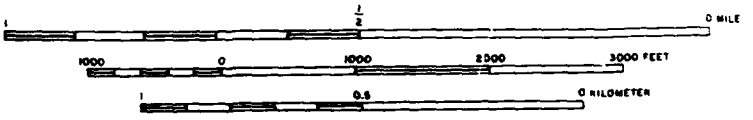
SEVEN OAKS DAM
CONCRETE MATERIALS SOURCES
UPPER SANTA ANA RIVER

SUBMITTED BY:
CHIEF, BRANCH
DATE APPROVED:
SPEC. NO. DACW09- -
DISTRICT FILE NO.
SHEET

PLATE A-66



PLAN OF EXPLORATION

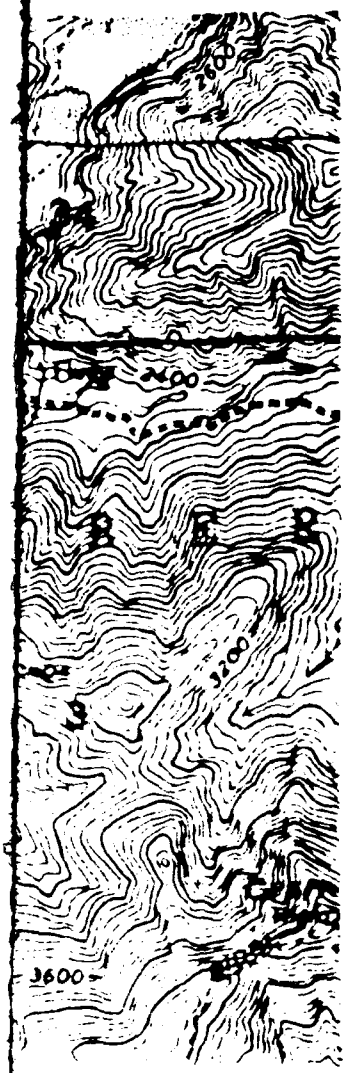


CONTOUR INTERVAL 40 FEET
NATIONAL GEODETIC VERTICAL DATUM OF 1929

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

S

LUE ENGINEERING PAYS



0 MILE

Notes:

1. Test trenches were excavated in August 1987 by a Caterpillar D-8H bulldozer.
2. Groundwater occurred only in TT 87-2 at a depth of 9 feet below the ground surface.
3. Samples were obtained from the spoils piles by excavation with a Caterpillar 977 tracked loader.
4. Gradations of materials are shown on plate A-98.

DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

SYMBOL	DESCRIPTION	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY: YBa
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

CONCRETE AGGREGATE INVESTIGATIONS,
LOCATION OF TEST TRENCHES

SUBMITTED BY: DATE APPROVED
CHIEF, ENGINEERING DIVISION

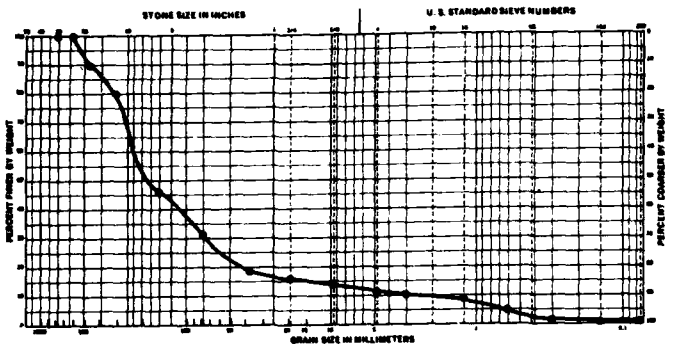
SPEC. NO. SACW 09-____-____
DISTRICT FILE NO.

SHEET

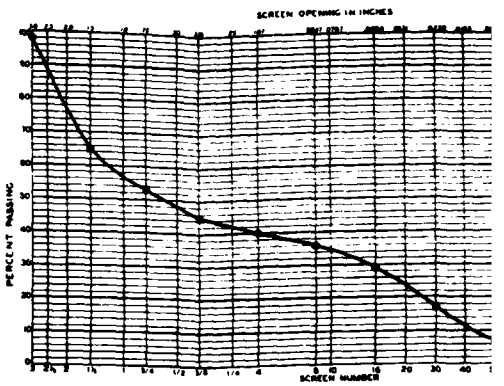
SAFETY PAYS

PLATE A-97

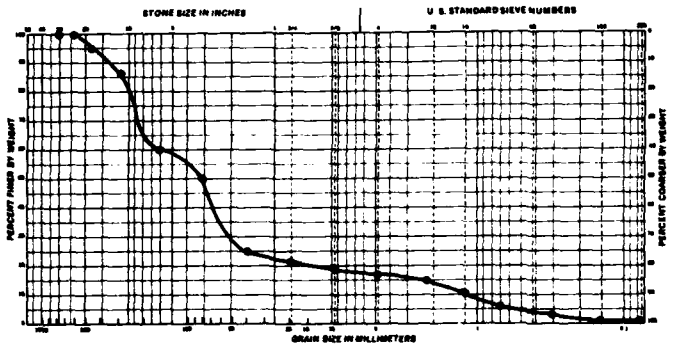
IN-SITU GRADATIONS
-3" MATERIAL



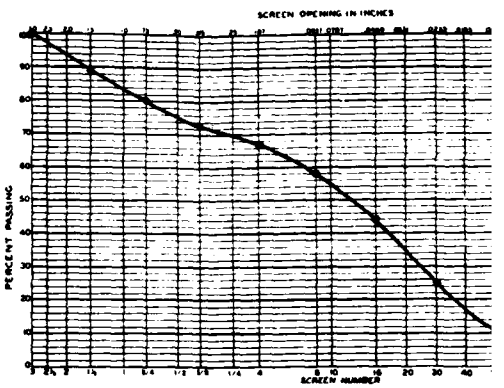
TT 87-1



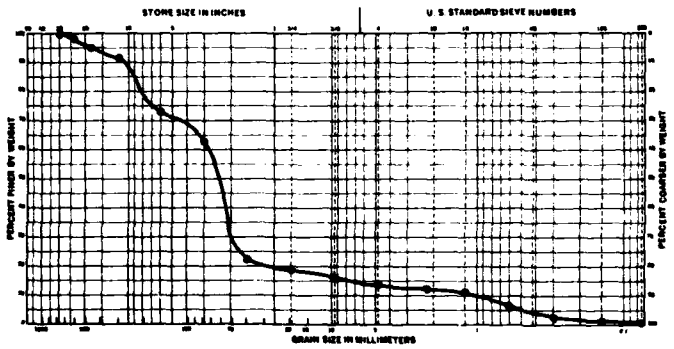
TT 87-1



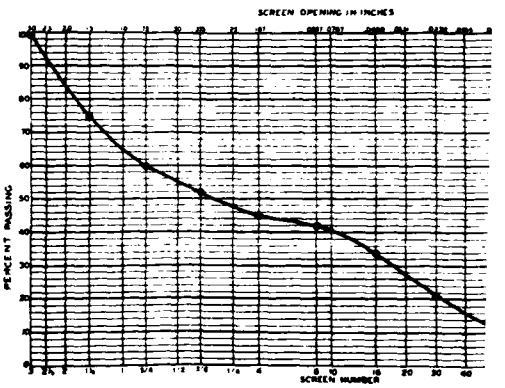
TT 87-2



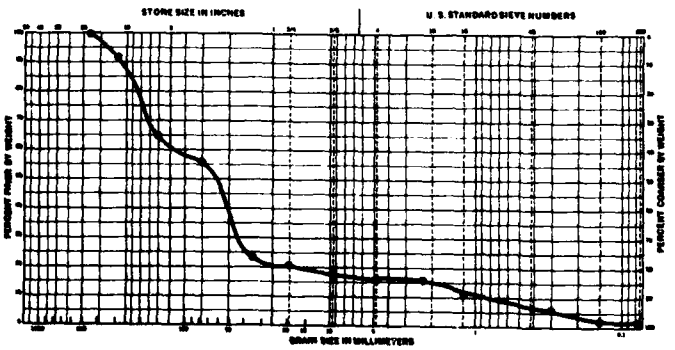
TT 87-2



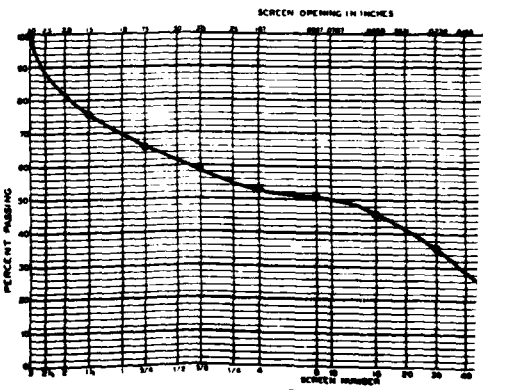
TT 87-3



TT 87-3



TT 87-4

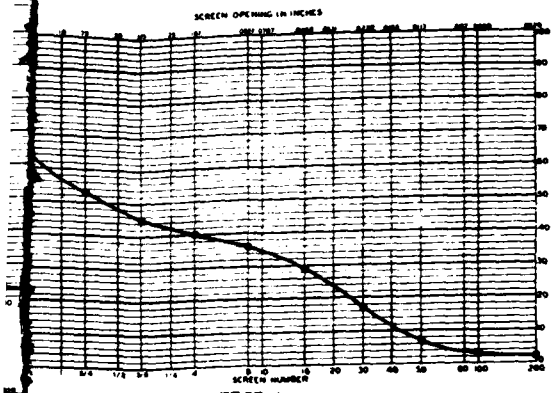


TT 87-4

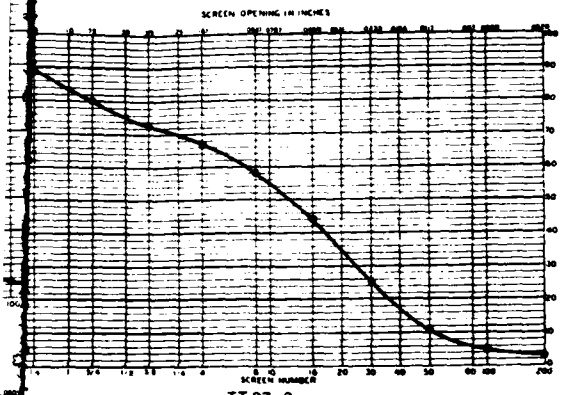
ENVIRONMENTAL
ENHANCEMENT
THRU ENGINEERING

VALUE ENGINEERING PAYS

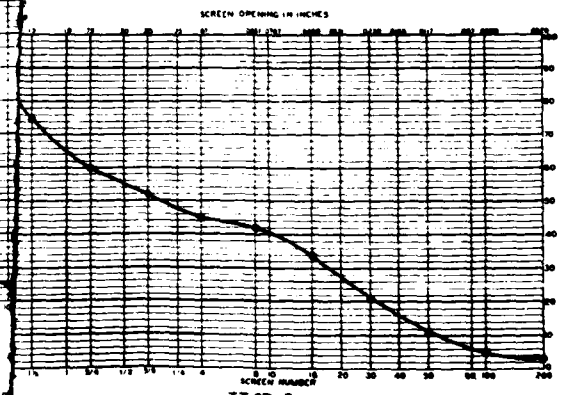
IN-SITU GRADATIONS
-3" MATERIAL



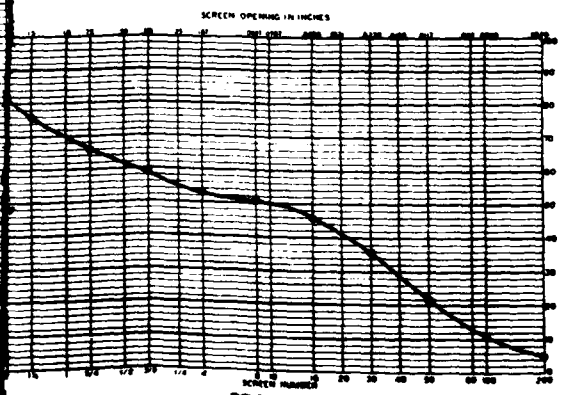
TT 87-1



TT 87-2

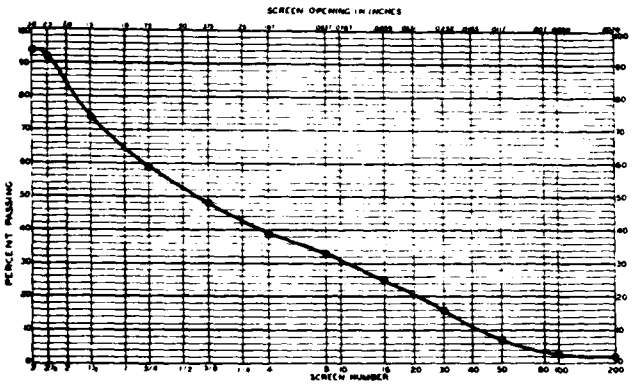


TT 87-3

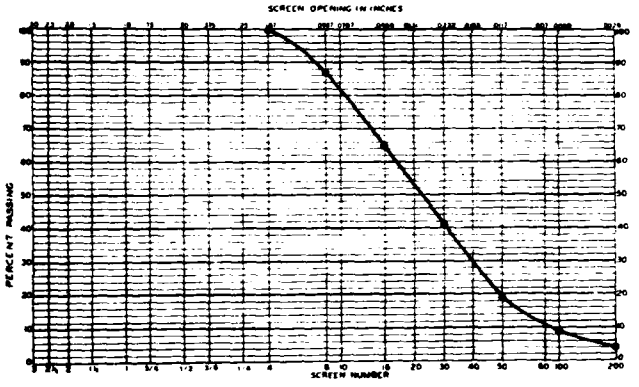


TT 87-4

CRUSHED PRODUCT
FROM TT87-3 AND TT87-4



COMPOSITE



UNWASHED SAND

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY: F.S.B. G.C.
CHECKED BY:

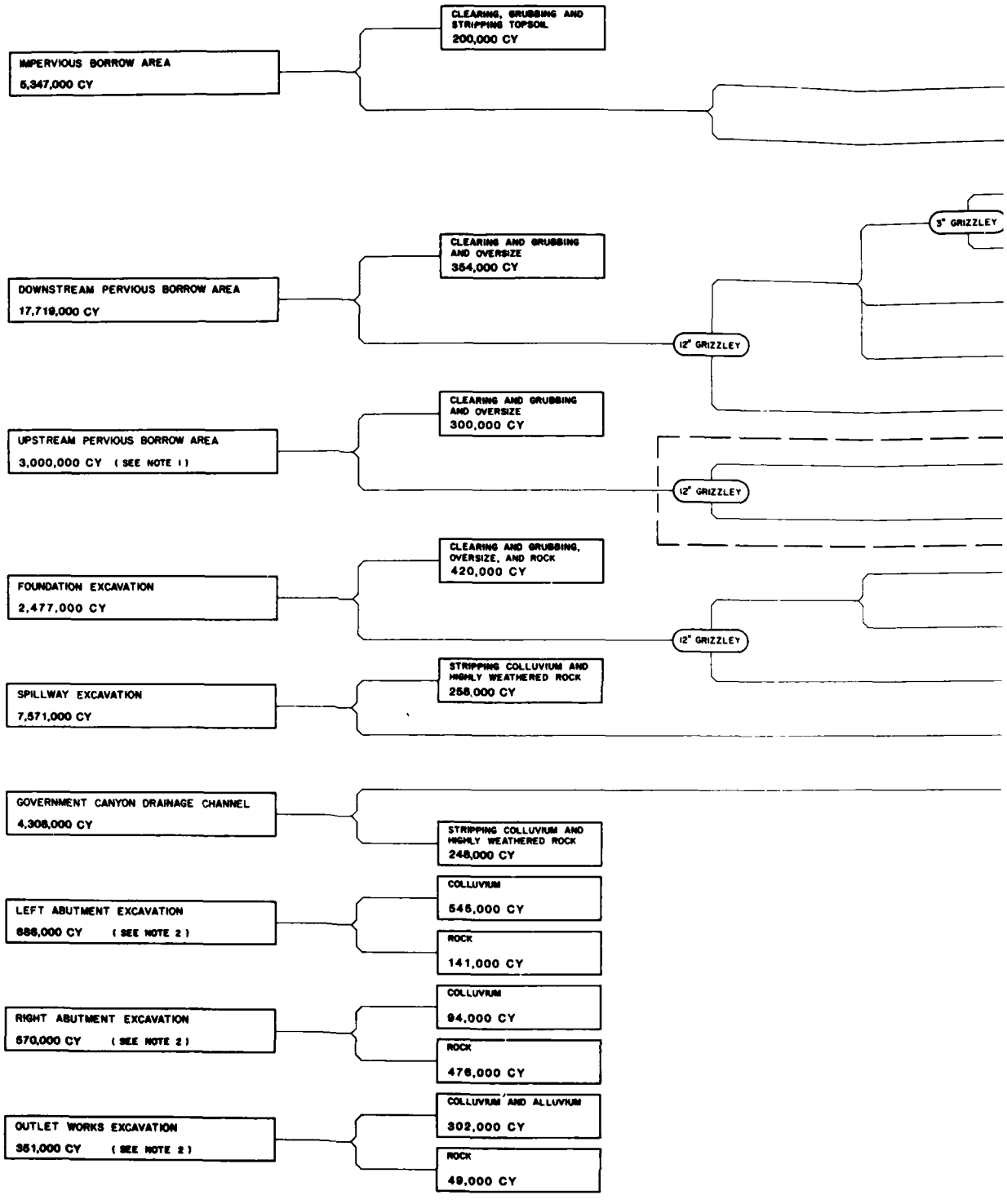
SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM
GRADATIONS FOR TT87-1 TO TT 87-4
AND
CRUSHED MATERIALS FROM TT87-3 AND TT 87-4

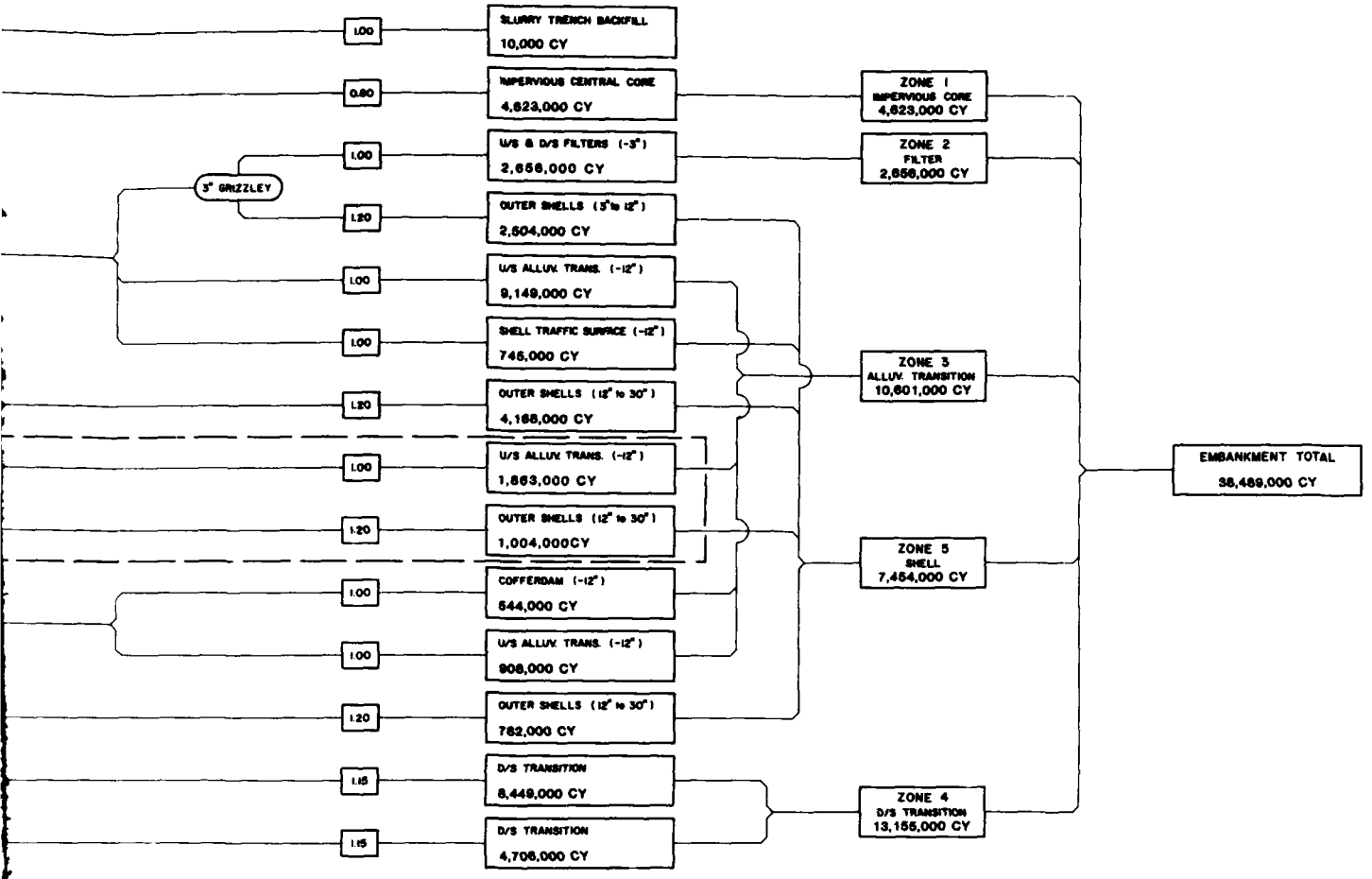
SUBMITTED BY: DATE APPROVED:
CHIEF, SURVEY
SPEC. NO. DACW09-___-___
DISTRICT FILE NO.
SHEET

PLATE A-98

SAFETY PAYS



ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING



NOTES:

1. DUE TO EXPECTED HIGH GROUNDWATER, USE OF UPSTREAM PERVIOUS BORROW AREA IS OPTIONAL. QUANTITIES FROM THIS SOURCE ARE NOT INCLUDED IN EMBANKMENT TOTAL SHOWN.
2. DISPOSITION OF MATERIALS EXCAVATED FROM ABUTMENTS AND OUTLET WORKS WILL BE DETERMINED DURING FDM STUDY PHASE AFTER ADDITIONAL INVESTIGATIONS AND TESTING.
3. SEE PLATE A-2 FOR LOCATION OF DAMSITE AND BORROW AREAS.
4. SEE PLATE A-80 FOR EMBANKMENT SECTION DETAILS.
5. SEE PLATE A-100 FOR QUANTITY VS. ELEVATION CURVES.

SYMBOL	DESCRIPTION	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

MATERIAL QUANTITIES AND DISTRIBUTION

SUBMITTED BY: DATE APPROVED:

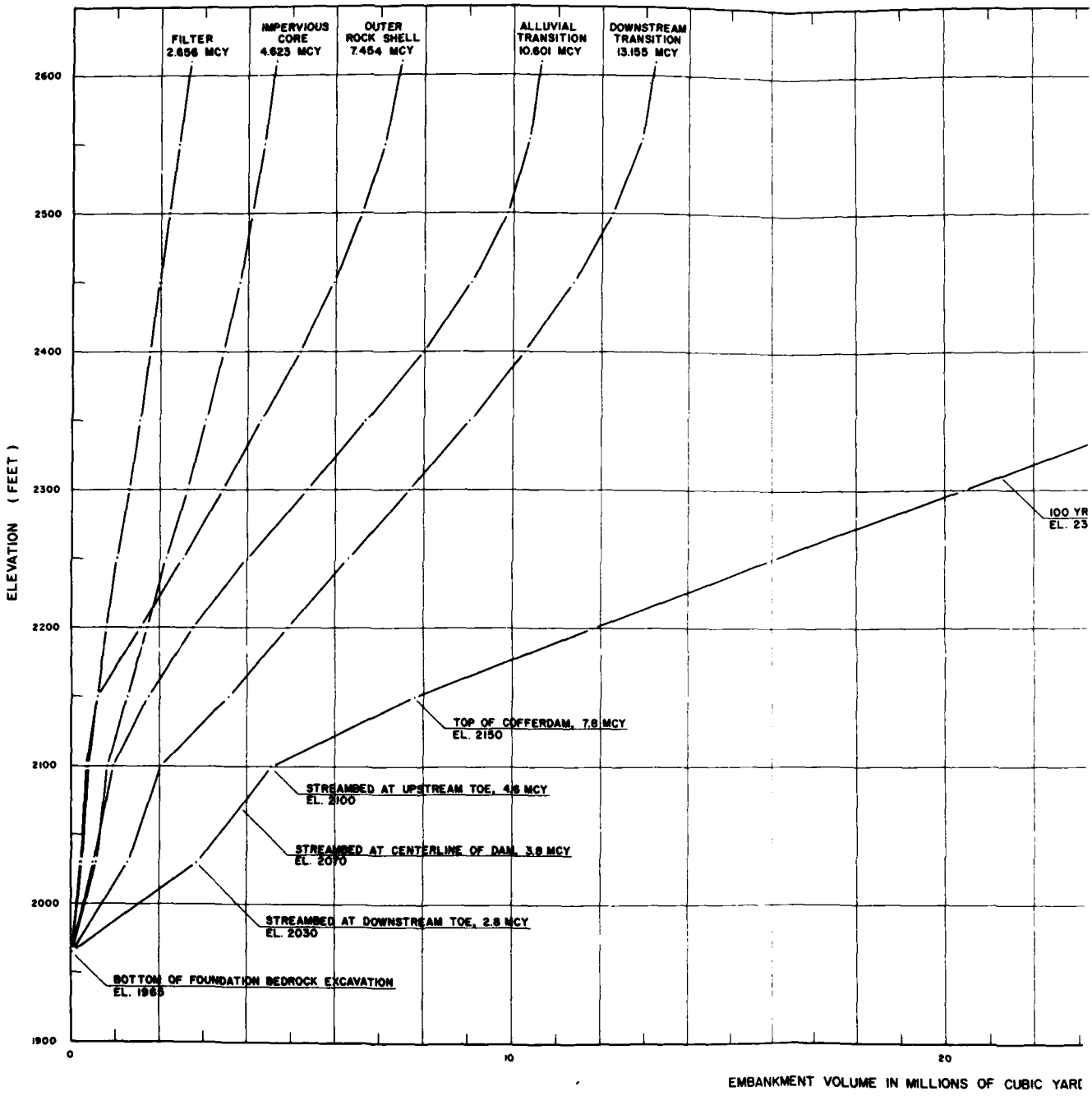
SPEC. NO. DACW09-____-____

DISTRICT FILE NO.

SHEET

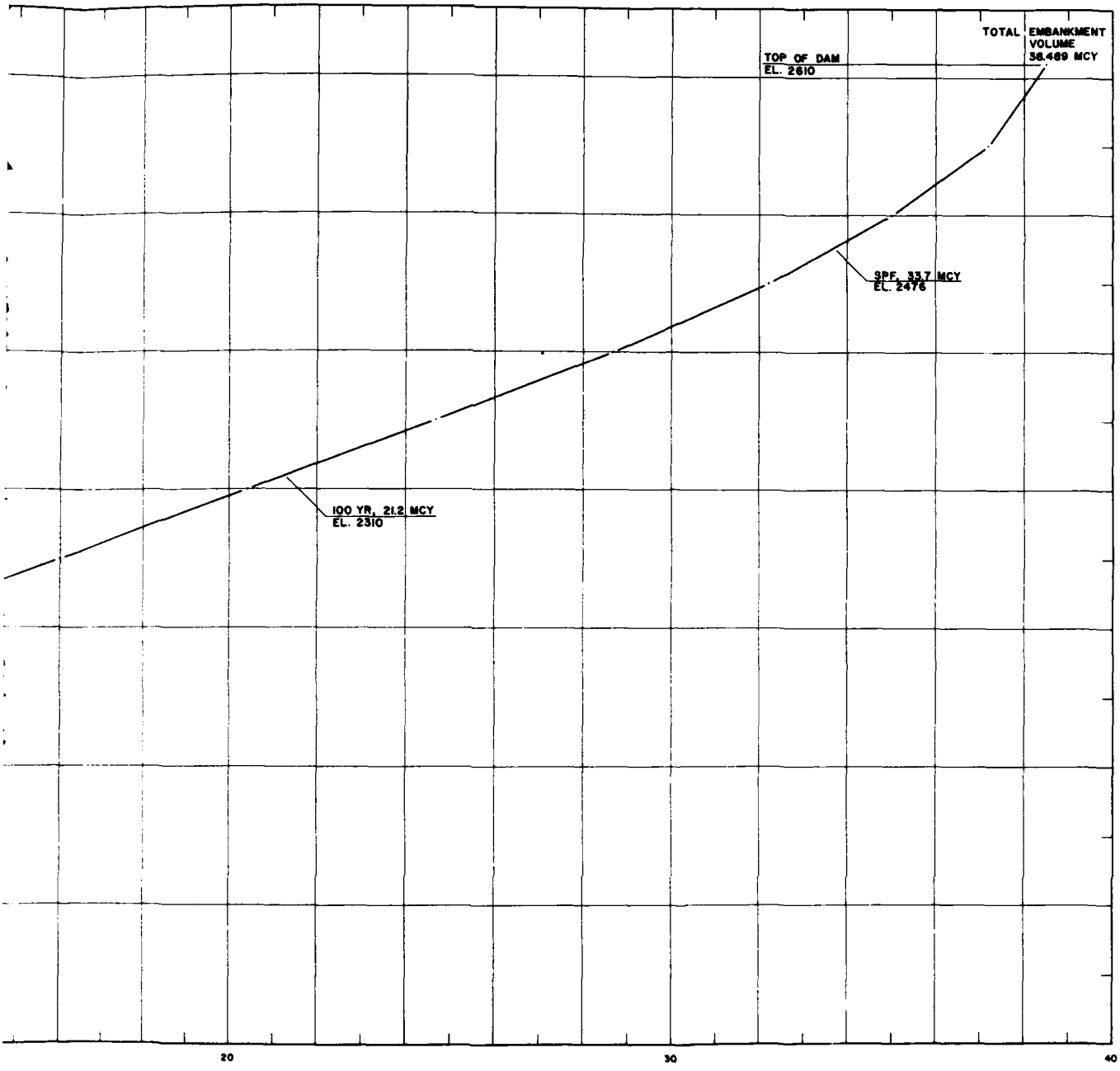
PLATE A-99

ENVIRONMENTAL
ENHANCEMENT
THRU ENGINEERING



NOTES:

1. SEE PLATE A-00 FOR EMBANKMENT SECTION
2. SEE PLATE A-00 FOR SOURCES OF EMBANKMENT MATERIALS.



EMBANKMENT VOLUME IN MILLIONS OF CUBIC YARDS (MCY)

- DATUM IS THE NATIONAL GEODETIC VERTICAL DATUM OF 1929

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

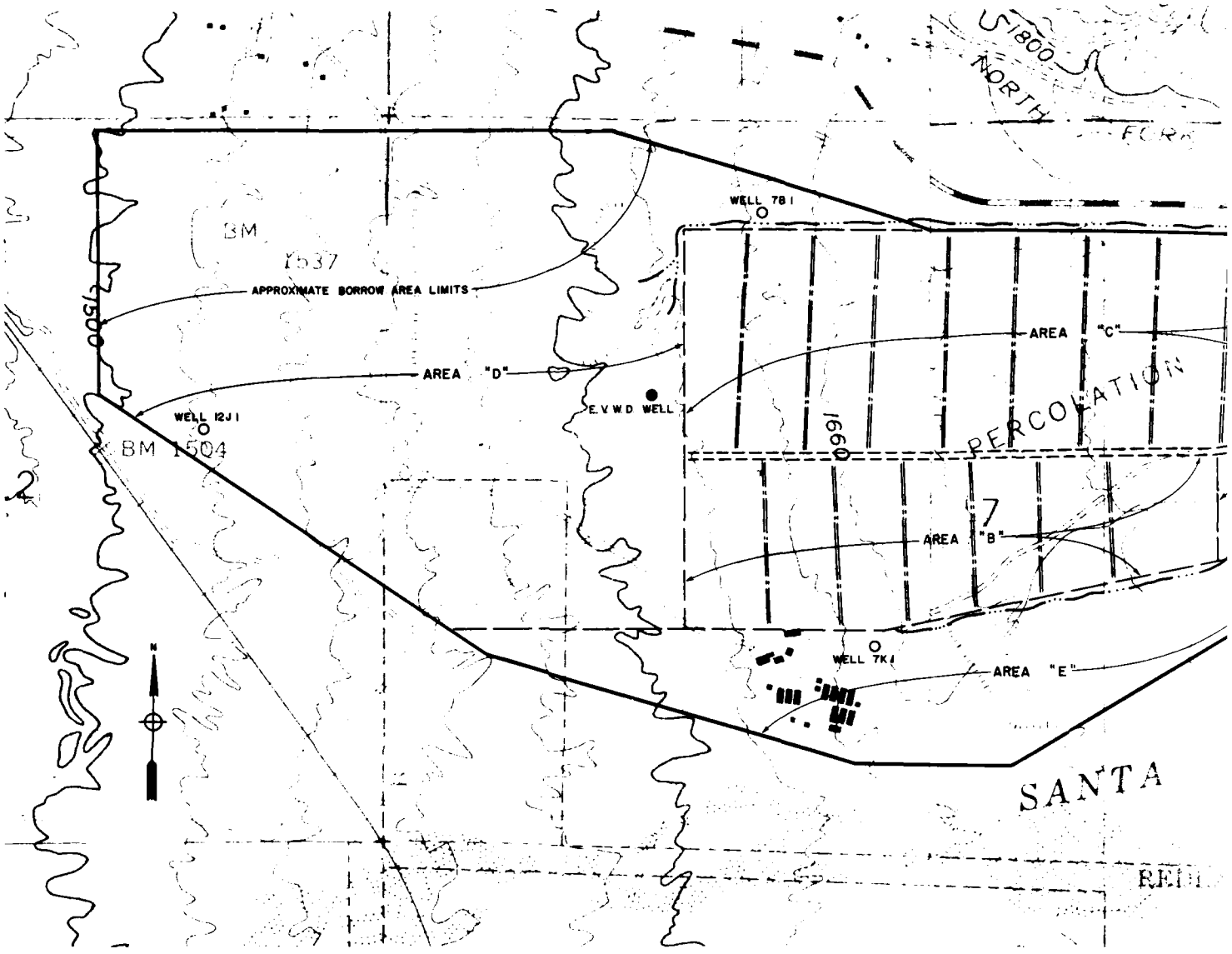
EMBANKMENT QUANTITIES VS. ELEVATION

SUBMITTED BY: DATE APPROVED:
CHIEF, ENGINEERING DIVISION

SPEC. NO. DACW 09- ___ B- ___
DISTRICT FILE NO.

SHEET

PLATE A-100



PROPOSED SEQUENCE OF BORROW EXCAVATION AND CONSTRUCTION REQUIREMENTS:

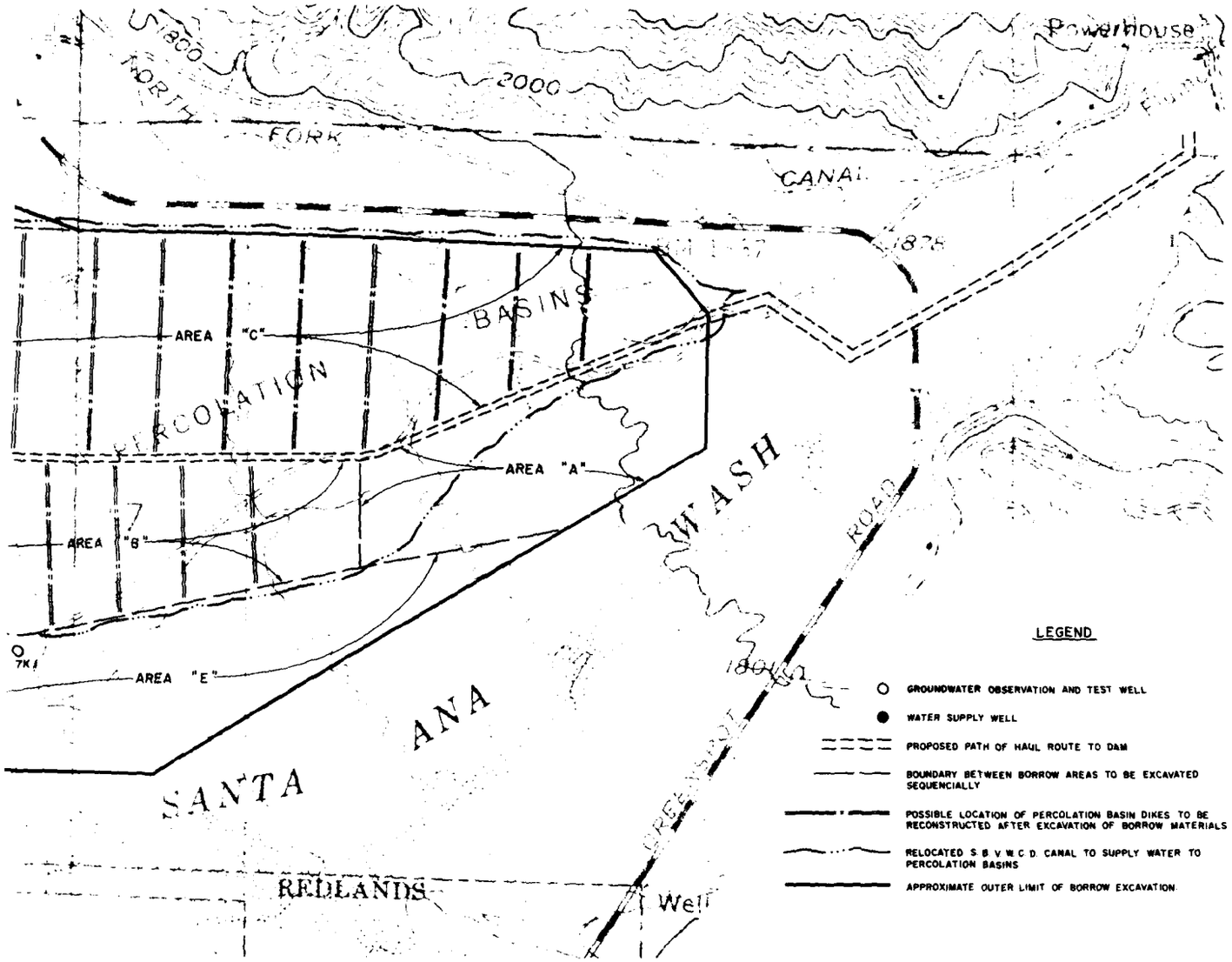
1. AREA "A" - EXCAVATE TO AVERAGE DEPTH OF 40 FEET AND USE AS DISPOSAL SITE FOR WASTE MATERIALS FROM EMBANKMENT AND BORROW AREA.
2. AREA "B" - EXCAVATE TO AVERAGE DEPTH OF 40 FEET, BEGINNING AT EITHER END, AND RECONSTRUCT PERCOLATION BASIN DIKES AS SHOWN.
3. AREA "C" - CONSTRUCT SUPPLY CANAL THROUGH AREA "A" TO DIVERT WATER TO AREA "B" PERCOLATION BASINS.
   - EXCAVATE TO AVERAGE DEPTH OF 40 FEET, BEGINNING AT WEST END, AND RECONSTRUCT PERCOLATION BASIN DIKES AS SHOWN.
   - CONSTRUCT SUPPLY CANAL ALONG NORTH EDGE OF AREA "C" TO DELIVER WATER TO PERCOLATION BASINS.
4. AREA "D" - EXCAVATE AS NEEDED TO SUPPLY MATERIALS FOR EMBANKMENT AFTER AREAS "A", "B" AND "C" ARE EXHAUSTED.
5. AREA "E" - EXCAVATE ONLY UNDER DIRECTION OF CONTRACTING OFFICER TO REDUCE IMPACTS ON BIOLOGICAL AND CULTURAL RESOURCES.

NOTES:

1 SEE F
2 SEE F OF FI
3. SEE F HAUL
4 SEE F FROM

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

ALUE ENGINEERING PAYS



(MENT AND BORROW AREA

I DIKES AS SHOWN

DIKES AS SHOWN

ED

AL RESOURCES

NOTES:

1. SEE PLATE A-2 FOR LOCATION OF BORROW AREA.
2. SEE PLATES A-55 THRU A-59 FOR PLAN OF EXPLORATION AND RESULTS OF FIELD AND LABORATORY TESTS.
3. SEE PLATE 24 (VOLUME I) FOR LOCATION OF UTILITIES ALONG PROPOSED HAUL ROUTE.
4. SEE PLATES A-99 AND A-100 FOR DISTRIBUTION OF PERVIOUS MATERIALS FROM BORROW AREA.

SCALE: 1 IN = 400 FT

500 0 500 1000 1500 2000

DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

SYMBOL	DESCRIPTION	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

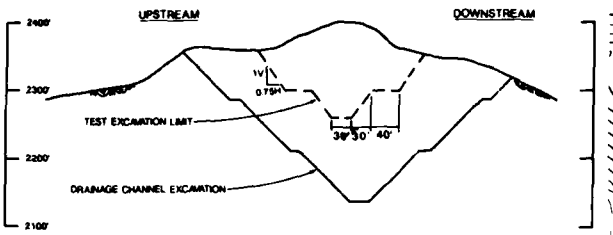
SEVEN OAKS DAM

DOWNSTREAM PERVIOUS BORROW AREA
CONSTRUCTION REQUIREMENTS

SUBMITTED BY: DATE APPROVED: SPEC. NO. DACW09- -B- SHEET
DIST. DISTRICT FILE NO.

PLATE A-101

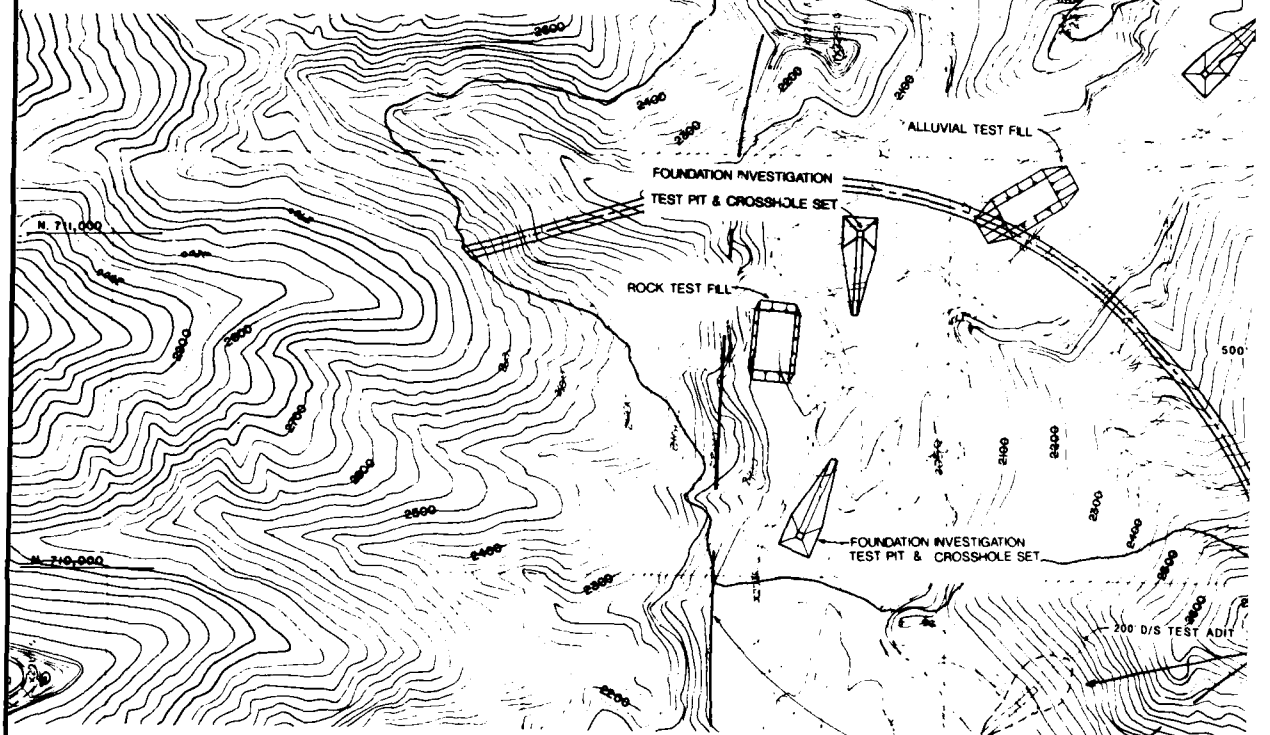
SAFETY PAYS

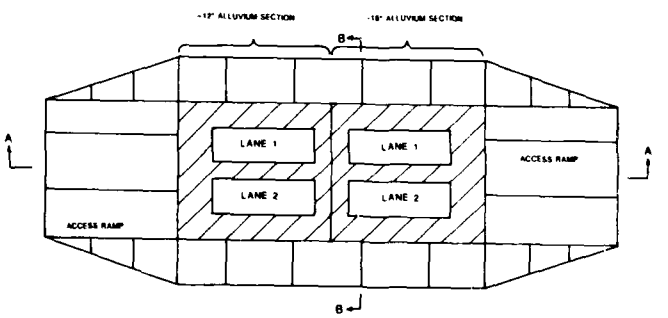


GOVERNMENT CANYON RIDGE

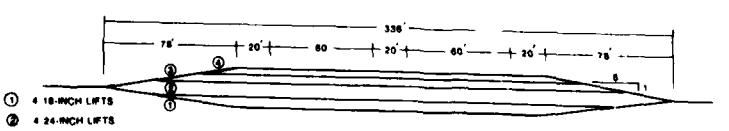
SECTION E - E

N.T.S

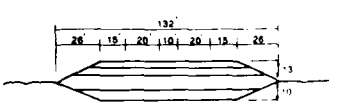




ALLUVIAL TEST FILL PLAN



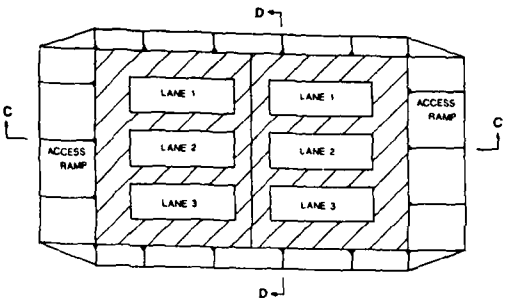
SECTION A-A



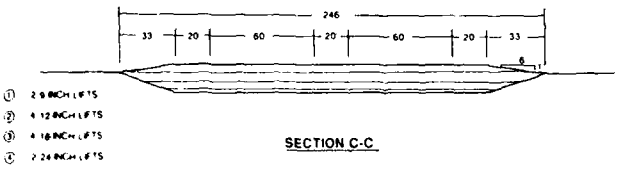
SECTION B-B

ENVIRONMENTAL
ENHANCEMENT
THRU ENGINEERING

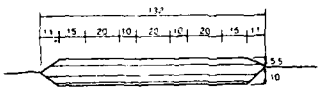
VALUE ENGINEERING PAYS



ROCK TEST FILL PLAN



SECTION C-C



SECTION D-D

NTS

FOUNDATION INVESTIGATION TEST PIT & CROSSHOLE SET

500' U/S TEST ADIT

APPROXIMATE LIMITS OF EXCAVATION FOR OUTLET WORKS ADIT INVESTIGATIONS

200' D/S TEST ADIT

APPROXIMATE LIMITS OF REQUIRED R.O.W FOR THE FIRST FEATUPE DESIGN INVESTIGATION CONTRACT

N 709,000

N 708,000

E 1,747,000

SCALE: 1 IN = 200 FT.

100 0 100 200 300 400 500

DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

SYMBOL	DESCRIPTION	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM
FEATURE DESIGN INVESTIGATIONS:
TEST FILLS, TEST PITS, TEST ADITS,
AND TEST EXCAVATION PLAN

SUBMITTED BY: DATE APPROVED:

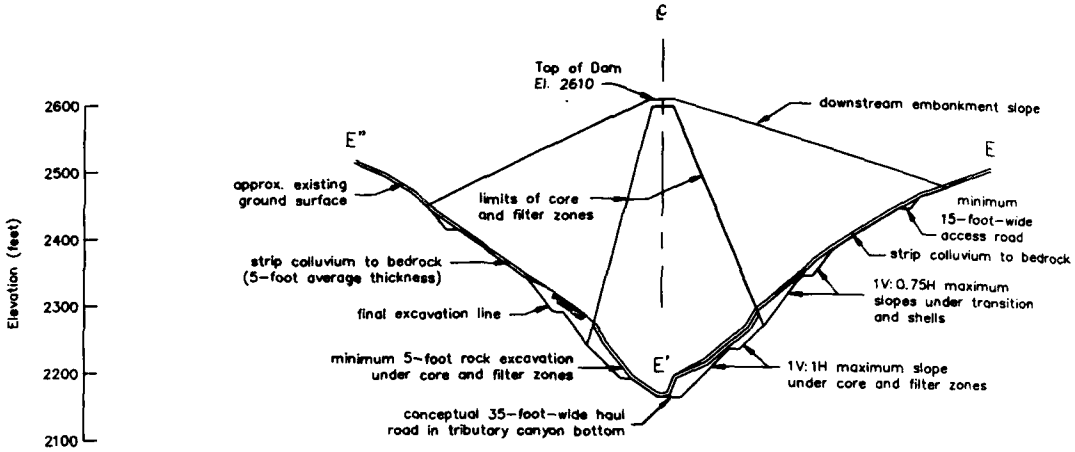
SPEC. NO. DACW09- -

DISTRICT FILE NO.

SHEET

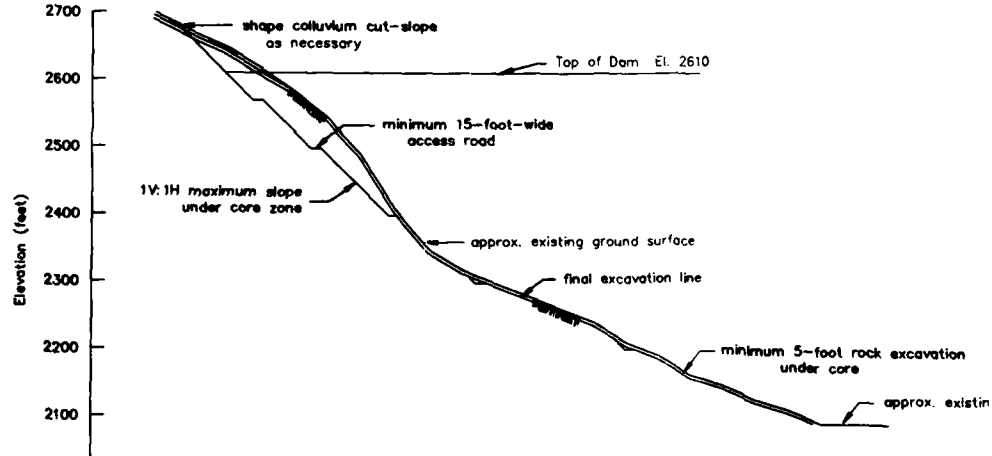
PLATE A-102

SAFETY PAYS



RIGHT ABUTMENT EXCAVATION
SECTION E-E"
(VIEW LOOKING EAST)

SCALE 100 50 0 100 200 300 FEET

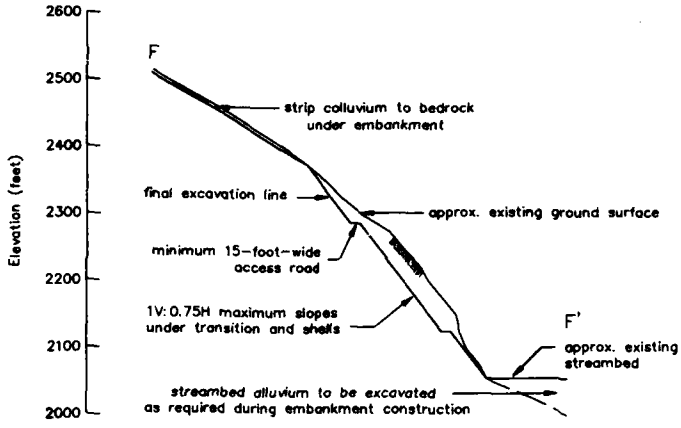


RIGHT ABUTMENT EXCAVATION
EMBANKMENT CENTERLINE PROFILE
(VIEW LOOKING UPSTREAM)

SCALE 100 50 0 100 200 300 FEET

ENVIRONMENTAL
ENHANCEMENT
THRU ENGINEERING

ALUE ENGINEERING PAYS

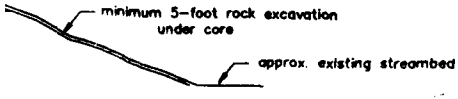


RIGHT ABUTMENT EXCAVATION
SECTION F–F'
(VIEW LOOKING UPSTREAM)

SCALE 100 50 0 100 200 300 FEET

NOTES:

1. See plate A–42 for location of sections.
2. Location of access–road cuts is conceptual only.
3. Sections and profile shown only for the right abutment because of the generally steeper topography. Identical excavation criteria apply to the left abutment.
4. Before placement of embankment core materials against the abutment, further shaping and scaling may be required as well as the smoothing of the cuts for the access roads.
5. See section 12 of this appendix for a discussion of abutment excavation criteria.



SYMBOL	DESCRIPTION	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

ABUTMENT EXCAVATION
PROFILE AND SECTIONS

SUBMITTED BY: DATE APPROVED:

SPEC. NO. DACW09-____-____

DISTRICT FILE NO.

SHEET

PLATE A–103

SAFETY PAYS

A GEOTECHNICAL

ATTACHMENTS

A GEOTECHNICAL

ATTACHMENT A-1

Assessment of Seismic Hazard
at the Upper Santa Ana River Damsite
by
Dr. Clarence R. Allen

CLARENCE R. ALLEN
1000 East California Blvd., Apt. 306
Pasadena, California 91106

8 December 1984

Mr. Norman Arno
Chief, Engineering Division
Los Angeles District
Corps of Engineers
P.O. Box 2711
Los Angeles, California 90053

Dear Mr. Arno:

This letter will report my conclusions concerning the seismic hazard of the Upper Santa Ana River damsite, following my visit with Mr. Lukesh of your staff to the area on July 24th and our conference in Los Angeles on October 16th. Needless to say, these are somewhat preliminary opinions, inasmuch as my visit was very brief, very little detailed geologic work has as yet been done in the area, and I have not had the opportunity to examine aerial photographs of most of the reservoir area. My opinions are based primarily on (1) our brief site visit, (2) my prior knowledge of the region dating back to the time of my Ph.D. thesis work in 1950-54 (Allen, 1957), (3) my continuing studies of southern California seismicity and seismic hazard (e.g., Allen et al., 1965; Allen, 1975; Crook et al., in press), and (4) my continuing involvement in studies of major damsites elsewhere in the world, particularly with regard to the problem of reservoir-induced earthquakes (e.g., Allen, 1982a). In addition, as you are aware, I was involved in the earlier discussions concerning the nearby Mentone damsite (Allen et al., 1975a; Allen, 1981, 1982b).

It must be recognized at the outset, of course, that the proposed damsite is very close to the San Andreas fault--about one mile away--and that a great earthquake of about magnitude 8 must be assumed to be capable of occurring on the adjacent segment of this fault during the service life of the structure. Indeed, my colleague Professor Kerry Sieh, on the basis of his paleoseismological field work in the Cajon Pass and Salton Sea regions, rates this particular segment of the San Andreas fault as the most likely to break in association with a great earthquake in the near future--with a probability of 50% to 90% within the next 50 years (Sieh, 1984; Caltech press release, 24 Sept. 1984). Insofar as I am aware, the Corps of Engineers is fully cognizant of this situation and is prepared to design the dam accordingly--as was also the case for the earlier Mentone site. Additional considerations at the Upper Santa Ana River damsite to which you wished me to direct my attention were (1) the presence of possibly active subsidiary faults even closer to, or beneath, the proposed structure and its associated

tunnels, and (2) the possibility of reservoir-induced earthquakes. In this connection, I will also comment below on further exploratory work that might help to resolve these concerns.

Although detailed geologic mapping by your staff has not yet commenced, two throughgoing faults close to the damsite have been identified that warrant concern. One of these, the Mill Creek fault (or North Branch of the San Andreas fault), passes about 3,000 feet north of the damsite and will cross a deep part of the reservoir. The preliminary geologic map prepared by your staff shows the Mill Creek fault in this area as lying beneath unbroken Pleistocene terrace gravels, which would imply that it is relatively inactive as compared to the nearby South Branch of the San Andreas fault, which multiply breaks much younger Holocene strata and produces a clear scarp and ground-water barrier in very young alluvium at the mouth of the canyon. A Caltech graduate student who has been doing geologic field work in this area, Ray Weldon, estimates (personal communication) that the age of this Pleistocene terrace is about 50,000 years. From what I could see in the field, I would tentatively agree that the terrace is unbroken, but one could and should look at this situation much more closely. However, even if the Mill Creek fault were considered to be active, the kind of shaking at the damsite that an earthquake on it might generate would be comparable to that which must already be considered from the nearby South Branch of the San Andreas fault, so that the question of the degree of activity of the Mill Creek fault is somewhat academic insofar as it affects the design of the proposed dam.

Traversing the downstream toe of the proposed dam is a second fault, informally termed the "Subsidiary fault" by your staff. It can be seen clearly in the west canyon wall, where it separates crystalline rocks of different types. Fortunately, it is apparently truncated here by an unbroken patch of Pleistocene terrace gravels about 120 feet above the canyon floor. This patch is a remnant of the same terrace as that which apparently truncates the Mill Creek fault farther upstream, so that this fault, too, can probably be considered relatively inactive--or at least not able in itself to generate a significant earthquake or sustain very large displacement.

As I understand the present plans, one concept being considered is a reservoir which would have a maximum water depth, with the Standard Project Flood and full conservation, of about 560 feet. This, however, would be a very temporary and rare situation, lasting for a few days or weeks at the most. The maximum sustained water depth, with full-time conservation storage, would be about 320 feet, which puts it in the category of a "deep" reservoir by worldwide standards (Packer at al., 1979; Stuart-Alexander, 1981; Baecher and Keeney, 1982). Therefore, in my opinion, there must be some concern for the possibility of significant reservoir-induced earthquakes, which was not the case for the Mentone damsite. Although the volume of water and debris to be stored at full conservation is about 100,000 acre feet, which is not large by world-wide standards, the correlation between induced earthquakes and water depth seems to be stronger than with volume, and even relatively small reservoirs with this depth of water have probably been associated with significant reservoir-induced earthquakes (Packer et al., 1979).

An added concern here is that the Mill Creek fault, which is a major geologic structure, traverses a deep part of the reservoir. Even if the Mill Creek fault is eventually deemed inactive, it clearly represents a through-

going zone of weakness, and the warm springs near the junction of Warm Springs Canyon and the Santa Ana River indicate that deep water circulation is currently taking place. Thus the Mill Creek fault, in my opinion, must be considered as the potential locus of a reservoir-induced earthquake (Allen, 1982), although the probability of this happening is very small, based on experience at other reservoirs. The maximum worldwide reservoir-induced earthquake to date was the 1967 magnitude 6.5 event at Koyna, India, and although still larger induced events are not physically impossible, planning for them seems unreasonable (Allen, 1982a). Thus the naturally occurring San Andreas earthquake would probably cause heavier shaking at the Upper Santa Ana River damsite than would any reasonable reservoir-induced earthquake, so that the possibility of a reservoir-induced event should not lead to additional design considerations except insofar as associated surface displacement on the Mill Creek fault might cause problems in itself.

However, the natural seismicity of this part of the San Bernardino Mountains is sufficiently high--with a 1935 earthquake of magnitude 5.1 having occurred perhaps on the Mill Creek fault nearby (Richter et al., 1958)--so that any earthquake in the vicinity of the reservoir might be claimed to be reservoir-induced and could not easily be proved otherwise. In all candor, we must admit that the state of the art is presently such that scientists would have difficulty in determining with confidence whether or not a given earthquake in the vicinity of the dam was in fact related in any way to the presence of the reservoir.

For the Mentone damsite, the Board recommended that 2 to 3 feet of fault displacement, in any direction, be considered credible on arbitrary subsidiary faults that might underlie the dam, because of its proximity to the San Andreas fault and because the local bedrock structure was concealed by alluvium (Allen et al., 1975a; Allen, 1981). The Upper Santa Ana River damsite is likewise close to the San Andreas fault, although the local bedrock is virtually 100% exposed. An added complication for the new site is that, because the San Andreas fault dips about 65° north at the mouth of the canyon, the site may be on the upthrown block of a thrust fault and is therefore in a somewhat more vulnerable location from the point of view of subsidiary faulting than are locations on the downthrown block to the south (Sherard et al., 1974). While it is true that the San Andreas fault probably steepens with depth, and it is certainly not a deep-seated thrust fault analogous, e.g., to the fault causing the 1971 San Fernando earthquake (Allen et al., 1975b), the San Bernardino Mountains may have been uplifted by a vertical component of displacement along the bounding San Andreas fault, and active splay and subsidiary faults appear to me more common north of the fault (within the mountains) than to the south. Furthermore, active strands of the San Andreas fault cannot be followed with complete continuity through the San Gorgonio Pass area to the east of the site, and it seems likely that a great earthquake in this region will be characterized by a certain amount of pervasive "shattering" of the southeastern San Bernardino mountain block, although predominantly along pre-existing faults and fractures.

A very recent and intriguing argument by Weldon and Humphreys (submitted for publication) and Weldon (1984) proposes that the relief along the south face of the San Bernardino Mountains, instead of being caused by a vertical component of displacement on the San Andreas fault, is caused simply by horizontal slicing apart by the San Andreas fault of a former combined San Bernardino-San Gabriel mountain mass. That is, the north-facing northern

escarpment of the San Gabriel Mountains is visualized as formerly juxtaposed against the south-facing southern escarpment of the San Bernardino Mountains, so that subsequent right-lateral strike-slip displacements on the San Andreas fault have simply sliced apart the two halves of the former single mountain mass without any significant vertical components of displacement. Supporting this point of view, according to Weldon and Humphreys, is the absence of evidence for current horizontal compression across the San Andreas fault in most of the San Bernardino-Cajon Pass area. While I agree that this argument is intriguing and probably at least in part correct, I am impressed with the northerly dip of the San Andreas fault at the mouth of the Santa Ana River Canyon, and I feel that a conservative approach demands that this be considered indicative of a vertical component of displacement. Furthermore, even Weldon and Humphreys agree that considerable current compressive stresses exist in the San Gorgonio Pass area to the southeast, where the fault traces are more complicated and where numerous active thrust faults are demonstrably present (Allen, 1957), and the Santa Ana Canyon area is sufficiently close to this area that it must share some of this same complex stress system.

With these factors in mind, I recommend designing the Upper Santa Ana River Dam on the assumption that as much as 4 feet of surficial fault displacement in any direction could take place arbitrarily beneath the facility, albeit as an exceedingly unlikely "maximum credible" event, and assuming about 300 feet of depth of permanent water storage. If the reservoir were instead to be nearly empty much of the time, a less conservative figure could perhaps be used. And if one could demonstrate that the local bedrock was completely unfractured, or that unbroken Pleistocene terraces covered the entire site area, this degree of conservatism might not be necessary, but such is not the case. As was recommended for the Mentone damsite (Allen, 1981), the specified fault displacement does not have to be assumed to pass directly through every localized structure of the dam (e.g., a spillway gate), but the basic integrity of the embankment and long tunnels--if critical to public safety--should be assured with this possible arbitrary displacement in mind.

In order to complete the initial seismic-safety analysis of the Upper Santa Ana River damsite, it is my opinion that several additional tasks should be undertaken by your staff:

(1) A high-quality geologic map of the entire reservoir area at 1:24,000 scale, emphasizing neotectonic relationships, should be produced, utilizing existing mapping but also undoubtedly requiring further field studies of your own or under contract. This is not a small task, but, in my opinion, experience at other reservoir sites, such as that at Auburn, demonstrates the wisdom of gaining a thorough understanding of the geological environment at an early stage in the project.

(2) As I mentioned to Mr. Lukesh in the field, it is essential that vertical aerial photographic coverage be obtained of the entire reservoir area for use in these studies. (Adequate coverage undoubtedly already exists at scales of about 1:20,000 and will not necessarily require a new flight contract.) Perhaps this has been accomplished since the time of my visit.

(3) A special investigation of the "Subsidiary fault" on the west wall of the Santa Ana River Canyon should be undertaken to document the apparent truncation of the fault by Pleistocene terrace gravels. This might require

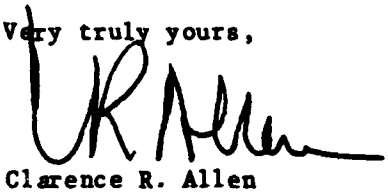
some cleaning of the contact, but the exposure looks relatively good already, at least from a distance.

(4) The suggested inactivity of the Mill Creek fault should also be the subject of a special field study, again involving relationships with terrace gravels. Since the Mill Creek fault is of some length, this study should include investigations of the fault outside of the immediate reservoir area as well as within it. I wouldn't be too hasty in accepting the conclusion that the Mill Creek fault is inactive. Although, as stated above, the degree of activity of the Mill Creek fault is a somewhat academic question in terms of the actual dam design, it is essential to have a thorough understanding of the tectonics of the reservoir area, and this fault is a key element within that tectonic pattern.

(5) Consideration should be given to increasing the seismographic coverage in the area. The nearest telemetered station of the USGS-Caltech network to the site is currently about 6 miles south at Crafton Hills (34° 02.11' N, 117° 06.66' W). Other nearby stations are at Mill Creek, about 10 miles east (34° 05.48' N, 116' 56.18' W), and at Butler Peak, about 11 miles north (34° 15.43' N, 117° 00.29' N). The nearest station to the west or northwest is at Cedar Springs Dam, more than 19 miles away. Certainly several years before construction starts, one or more telemetered stations should be established in the immediate reservoir area.

(6) Although Ray Weldon, the Caltech graduate student who has been carrying out field work in this area, is now committed to the U. S. Geological Survey and therefore apparently cannot formally consult for the Corps of Engineers, I urge that you communicate with him about his ideas on the structure and geomorphology of the region. Since these studies were a part of his Ph.D. thesis at Caltech, he should have no hesitation in talking about them.

Very truly yours,



Clarence R. Allen

cc: Mr. David Lukesh

REFERENCES

Allen, C. R., 1957, San Andreas fault zone in San Gorgonio Pass, southern California: Geol. Soc. America Bull., v. 68, p. 315-350.

Allen, C. R., 1975, Geologic criteria for estimating seismicity: Geol. Soc. America Bull., v. 86, p. 1041-1057.

Allen, C. R., 1981, Letter to Mr. Arno, dated 2 January 1981.

Allen, C. R., 1982a, Reservoir-induced earthquakes and engineering policy: California Geology, v. 35, p. 248-250.

Allen, C. R., 1982b, Letter to Mr. Arno, dated 2 February 1982.

Allen, C. R., Bolt, B. A., Newmark, N. M., and Seed, H. B., 1975a, Letter (Consulting Board report) to Col. Foley, dated 11 March 1975.

Allen, C. R., Hanks, T. C., and Whitcomb, J. H., 1975b, Seismological studies of the San Fernando earthquake and their tectonic implications: California Div. Mines and Geology Bull. 196, p. 257-262.

Allen, C. R., St. Amand, P., Richter, C. F., and Nordquist, J. M., 1965, Relationship between seismicity and geologic structure in the southern California region: Seismol. Soc. America Bull., v. 55, p. 753-797.

Baecher, G. B., and Keeney, R. L., 1982, Statistical examination of reservoir-induced seismicity: Seismol. Soc. America Bull., v. 72, p. 553-569.

Caltech press release, 24 Sept. 1984: San Andreas faults yielding its secrets.

Crook, R., Allen, C. R., Kamb, B., Payne, C. M., and Proctor, R. J., in press, Quaternary geology and seismic hazard of the Sierra Madre and associated faults, western San Gabriel Mountains, California: U. S. Geol. Survey Prof. Paper.

Packer, D. R., Cluff, L. S., Kneupfer, P. L., and Withers, R. J., 1979, Study of reservoir induced seismicity, Woodward-Clyde Consultants, for the U. S. Geol. Survey, contract no. 14-08-0001-16809.

Richter, C. F., Allen, C. R., and Nordquist, J. M., 1958, The Desert Hot Springs earthquakes and their tectonic environment: Seismol. Soc. America Bull., v. 48, p. 315-337.

Sherard, J. L., Cluff, L. S., and Allen, C. R., 1974, Potentially active faults in dam foundations: Geotechnique, v. 24, p. 367-428.

Sieh, K., 1984, Lateral offsets and revised dates of large prehistoric earthquakes at Pallett Creek, southern California: Jour. Geophys. Research, v. 89, p. 7641-7670.

Stuart-Alexander, D. E., 1981, Reservoir induced seismicity: U. S. Geol. Survey Open-File Rept. 81-167, p. 461-464.

Weldon, R., 1984, Quaternary deformation due to the junction of the San Andreas and San Jacinto faults, southern California [abstract]: Geol. Soc. America Abtsracts with Programs, v. 16, p. 689.

Weldon, R., and Humphreys, G., submitted for publication, A kinematic model of southern California: Tectonics.

A GEOTECHNICAL

ATTACHMENT A-2

Seismological Report for Seven Oaks Dam
for Santa Ana River Basin, California
by
Bruce A. Bolt

I. Introduction and Scope of Work

An assessment is now underway of alternatives for Upper Santa Ana River flood storage by the U.S. Army Corps of Engineers, Los Angeles District. An important part of this study is a quantitative estimation of geotechnical parameters required for the seismic evaluation of the foundation for either an earthfill embankment dam or a roller compacted concrete dam at the Seven Oaks Dam site.

This dam site is located approximately 2 km upstream from the mouth of Santa Ana River Canyon near the south branch of the San Andreas fault. According to the Phase I Assessment provided by the Corps of Engineers, the North Branch of the fault crosses the reservoir area about 1 km upstream from the proposed dam site. The rock wedge between the two branches of the San Andreas fault in this vicinity is composed of Pre-Cambrian gneiss and Cretaceous diorite. The rock is often highly sheared and fractured coves from exploratory holes indicate that the rock is fractured to depths of several hundred feet, with evidence from thin clay gouge slickenslides of movement by fault slip. At the dam site, the Santa Ana River has eroded a symmetric canyon into the fractured and weathered bedrock.

As requested, this report addresses the following seismological questions.

A. Provision of seismic parameters for design of a zoned earth-rock-filled embankment dam at the subject site including the following:

1. Properties of the seismic fault sources
2. Distances between seismic sources and the dam site

3. Earthquake activity rate

    a. Recurrence interval

4. Definition of the maximum credible earthquake

    a. Magnitude and seismic moment

    b. Intensity and maximum ground acceleration, velocity and displacement at the site

5. Definition of the maximum probable earthquake

    a. Magnitude and seismic moment

    b. Intensity characteristics such as maximum ground acceleration, velocity, and displacement at the site

6. Acceleration time history

7. Bracketed duration above 0.05g

8. Predominant period

B. Provision of any differing seismic parameters from Case A for design of roller compacted concrete dam at this site.

C. Provision of comments on the proposed design measures (joint spacing and separation at the foundation contact), addressing the postulated 4-foot displacement in the foundation and abutments of the roller compacted concrete dam.

II. Seismicity and Active Fault Information

The seismicity of this area near the San Bernardino Mountains (between the San Andreas, San Jacinto and Banning faults) has been summarized in the Phase I Assessment (D-14 to D-16) provided by the Corps of Engineers (31 December, 1986). Earthquake occurrence has also been addressed in an attachment by Clarence R. Allen (Letter 12/8/1984). Historically, and at present, the area has been the site of numerous small to moderate earthquakes (see Figure 7, Ziony, 1985) with the largest within 15 km of the site being a 1935 earthquake of magnitude 5.1, perhaps caused by slip near the Mill Creek fault. Although no great earthquakes have occurred in historical times from seismic sources near the San Bernardino Mountains, the geodetic , geological and seismological evidence together is quite convincing that earthquakes with magnitudes up to 8+ could be produced by elastic rebound of long sections of the major faults in the area.

Specifically, the site lies between the North Branch and the South Branch (San Bernardino segment) of the San Andreas fault. The Phase I Assessment quotes recent geological studies that it is likely that the presently active trace is the South Branch (about 2 km from the site). Professor Allen has discussed the evidence for Quaternary slip on the North Branch (Mill Creek fault), located about 1 km from the site. It is clear that the assessment of seismic intensity for dam design purposes would not differ for major fault slip on either Branch; for a magnitude 8+ earthquake the dam, site is in the near seismic field of either fault source.

III. Definition of Seismic Sources

As discussed in the last section, given the tectonic and seismic history of the area, the maximum credible earthquake for this site is one similar to the 1857 Fort Tejon earthquake on the southern San Andreas fault and the 1906 earthquake on the northern San Andreas fault. Such earthquakes are produced by long fault rupture (up to 430 km) with offsets of many meters, mainly in a right-lateral strike-slip sense. The magnitude of the 1906 earthquake was M_s = 8.25 as measured by seismographs recording in Europe.

A statistical estimate based on recorded seismicity for the recurrence time of the "maximum credible earthquake" is not feasible because of the short historical interval (going back approximately to 1800). A useful estimate can, however, be obtained from the newly developed geomorphological and stratigraphic methods of inferring average time between major earthquakes produced by slip on the San Andreas fault. The evidence acquired by G. Sieh (1978) in the Pallet Creek area north of this site indicates that there is a recurrence interval of 130 to 150 years between earthquakes large enough to cause liquefaction of sand and peat layers in that area. The main uncertainty is in the magnitude associated with each event and not all the earthquakes in the set treated by Sieh might reach magnitudes in excess of 8. Nevertheless, a value of 150 years is clearly appropriate as an interoccurrence time for an 8+ earthquake that would be produced by major slip of one or other branch of the San Andreas fault near the dam site. Because no such earthquake has occurred for almost 200 years it must be regarded as a likely event.

The estimation of a "maximum probable earthquake" is less straight-forward than that of the "maximum credible earthquake" because, as mentioned above, the time series of recorded earthquakes in the area is short. My approach to the problem is to consider a reasonable life expectancy for the dam structure and consider the greatest earthquake to likely occur within this time. A reasonable project operation interval is 50 years, say. As pointed out in the Phase I Assessment (p. D-64), various realistic published earthquake recurrence models predict that a 8+ magnitude event is "overdue" along the southern San Andreas fault. Thus, in terms of a 50 year time window, the maximum credible earthquake is also close in size to the maximum probable earthquake in this case. The seismic moment of such an earthquake is, from measurements in similar cases elsewhere, about 10^{28} dyne-cm.

The above argument is supported by work by U.S.G.S. in Southern California (Ziony, p. 80, 1985). The published conclusion is "the probability of occurrence of 1857-type earthquakes along the (southern San Andreas) fault is currently estimated to be about 1 percent per year but about 40 percent within the next 30 years. Thus an earthquake of M_s = 8 appears credible for ordinary planning and design purposes".

It is not necessary, however, for the main release of seismic energy from the fault dislocation to be produced at the nearest part of the San Andreas fault to the site. A more probable distance is 20 km (see Table 1).

IV. Estimation of Ground Motion Parameters

A. Zoned Earth-Rockfill Embankment Dam

In the previous section, the maximum credible earthquake was defined as a magnitude 8+ event produced by extensive right-lateral rupture of the nearby San Andreas fault. Surface fault ruptures involving elastic rebound or fault fling could occur within 2 km of the dam site in such a major dislocation. By extrapolation from similar major earthquakes such as the 1906 San Francisco earthquake, the major rupture may extend for over 200 km with horizontal surface offsets of up to 5 meters along the main fault trace. Given this seismic source, estimation of the strong ground intensity at the Seven Oaks dam site must proceed by extrapolation using seismological theory and ground motion measurements obtained by accelerographs in similar source and site situations. The construction is not direct because no records of strong motion have yet been obtained near a shallow right strike-slip faulting producing an earthquake with a magnitude $M_s > 7.5$.

The estimation method followed here to compute the requisite seismic intensity parameters and an appropriate synthetic strong motion accelerogram is as follows. The resulting values are listed in Table 1. First, the peak ground acceleration velocity and displacement are selected from the most appropriate recently published attenuation curves (e.g. Joyner and Boore, 1985, Seed and Idriss, 1985, Abrahamson and Bolt, 1982). The magnitude dependence of expected peak ground parameters very near to the fault source is not well known because of lack of large-earthquake data. Nevertheless, the limited elastic strength and rigidity of the strained rocks place limitations on the values of the supremum amplitudes (i.e. peak maxima of the waves). In addition, the depth of the slip

TABLE 1

Estimated (Mean) Ground Motion Parameters on Rock at Site

	Max. Credible Earthquake (Adjacent San Andreas fault)	Max. Probable Earthquake (in 50 years)
Source Distance to Dam Site (km) (Surface Rupture)	2	20
Magnitude (M_s)	8+	7.5 - 8.0
Seismic Moment (dyne-cm)	10^{28}	10^{27}
Recurrence Rate	150 ± 30 years	1 percent chance per year
Peak Horizontal Acceleration	0.70g	0.5g
Peak Horizontal Velocity (cm/sec)	90-105	70-80
Peak Horizontal Displacement (cm)	50-80	40-70
Bracketed Duration at 0.5g (sec)	40-50	35
Predominant Period in ground velocity (sec)	0.2-10.0	0.3-8.0

surface where most seismic energy is released (below about 5 km in the San Andreas case) and the high wave attenuation in the fractured fault zone place a bound on the local peak accelerations. Consultation of empirical attenuation curves which extrapolate to higher magnitude suggest that an expected peak ground acceleration of 0.7g is an appropriate estimate in the present case. It is emphasized that the value is a mean value at a rock site. The effect of weathering and widespread cracking of the rock would be to reduce this estimate slightly. Similar correlations give peak horizontal velocities of the ground in the range of 90-105 cm/sec and peak ground displacements of 50-80 cm. Empirical curves (Bolt, 1973) for the bracketed duration of shaking above an acceleration of 0.05g yield an interval of 40-50 secs. For a magnitude 8+ earthquake there will probably be a persistent train (or "coda") of long-period (greater than 5 sec) surface waves with accelerations below 0.05g.

The predominant period window of the ground motion given in Table 1 is estimated from sets of amplitude spectra for strong ground motions available in California Strong Motion Instrumentation Program analyses and other literature (e.g. Seed and Idriss, 1982). Both Fourier spectra and response spectra (with small damping) indicate that for the largest earthquakes (M_s > 7.0) in the near field, maximum velocity ground response is in the range 0.2 to 10.0 sec (5 to 0.1 Hz) for (weathered) rock sites. At longer periods, near to a major fault rupture some enhancement of ground motions is expected (see discussion Section V).

Finally, parameters are also given in Table 1 for the dam site in the case of a maximum probable earthquake as defined in an earlier section.

The estimates have been obtained in the same way as for the maximum credible earthquake. There are only small differences in values, probably within the uncertainty (as measured by a reasonable standard deviation) for these parameters. The differences arise from adopting a greater distance for the source-to-site distance and a smaller seismic moment.

B. Roller Compacted Concrete Dam

The scope of work specifies that consideration be given to any differences in ground motion that should be considered for construction of a dam of the above type compared with construction of an earth rock-fill dam.

The strong ground shaking parameters estimated for the latter in IV-A above (see Table 1) were derived using field measurements from strong motion accelerometers for a rock site at an appropriate source-site distance. The values are independent of the type of engineering structure being considered. In particular, occurrence rate and size for both the maximum credible and maximum probable earthquakes are the same for the two types of dam being contemplated for Seven Oaks Dam.

Similarly, the ground motion intensity parameters were estimated from regressions of observed values, conditioned on appropriate ranges. In applications to design considerations, however, differences might arise due to different methods of structural analysis. The frequency range of the input motions must be appropriately broad to accommodate the full modal response of both types of dam. The peak parameter values given in Table 1 apply to waves in the range 5-10 Hz which is normal for scaling time histories and response spectra. Similarly, the bracketed duration applies to a spectral window with an upper bound of about 10 Hz.

Attention must also be given to the frequency range of uniform spectral power (wave amplitude squared) in time histories adopted for design purposes. This spectral uniformity is provided in the synthetic accelerogram provided in the next section. With this proviso, there appears to be no reason to modify the values in Table 1 for application to a roller-compacted concrete dam.

V. Synthetic Ground Acceleration Time-History

Because no field recordings in the near field of a M_s = 8+ strike-slip shallow-focus earthquake exist, a synthetic accelerogram must be constructed which is consistent with seismological theory of the appropriate source mechanism and propagation path. Kinematic scaling is carried out on ground acceleration, velocity and displacement and spectral content by extrapolation from a selected set of smaller earthquake recordings.

Several seismological aspects of the ground motion can be set down as criteria. First, the earliest part of the shaking consists mainly of P and back-scattered S waves. The requirement that the source produce the postulated maximum credible earthquake at the site suggests that the interval between the S and P onsets be from 3 to 5 secs (i.e. the focus no more than 25 km away). Secondly, because the site is close to the rebounding fault, the ground motion should contain a pulse at about the time of the major S wave arrival that models the fling of the fault as the rupture goes by the site. This longer period fling is of importance in a number of engineering contexts. A displacement SH pulse with periods of several seconds will propagate outwards and although not necessarily having the largest acceleration in the wave train, it may be associated with the greatest kinetic energy. For example, from studies of the damaged Olive View Hospital in the 1971 San Fernando earthquake (Bertero et al., 1978), failures in that structure apparently occurred during such a long duration pulse. It is now regarded as good practice to include in the appropriate portion of the near source record such longer period pulses to ensure that the longer period responses of a structure are realistic.

Further, in developing a synthetic time-history appropriate for this near-field site, there must be realistic constraints on the spectral components of the motion. This problem is treated by comparing pseudo-relative velocity spectra from seismic ground motions previously observed with that calculated for the synthetic record. The seismological proviso is however, that the synthetic record should have a response spectrum with somewhat higher spectral amplitudes in the long period range because of the longer fault slips and rise times associated with large magnitude earthquakes.

Few records which meet the above criteria are available in the published literature. The time history recommended as appropriate for the maximum credible earthquake is shown in Figure 1. It was numerically constructed by the method of Fourier decomposition (see Bolt, 1981), first for a major strike-slip earthquake on the Hayward fault, and later substantially modified (in conjunction with Dr. J. Valera of Earth Sciences Associates for a study by the City of San Francisco) for the near field a magnitude 8+ earthquake produced by rupture of the San Andreas fault in central California.

Figure 1 shows the horizontal component seismograms for acceleration, velocity and acceleration as scaled to satisfy the site-specific parameters of Table 1. The S minus P interval is appropriate for rupture commencing near to the dam site and a pulse-like displacement is incorporated in the first S wave packet. Longer period surface waves are contained in the coda and the most important 62 sec of shaking are provided.

The recommended synthetic time history has been checked to satisfy the spectral requirements mentioned above. First, the Fourier velocity

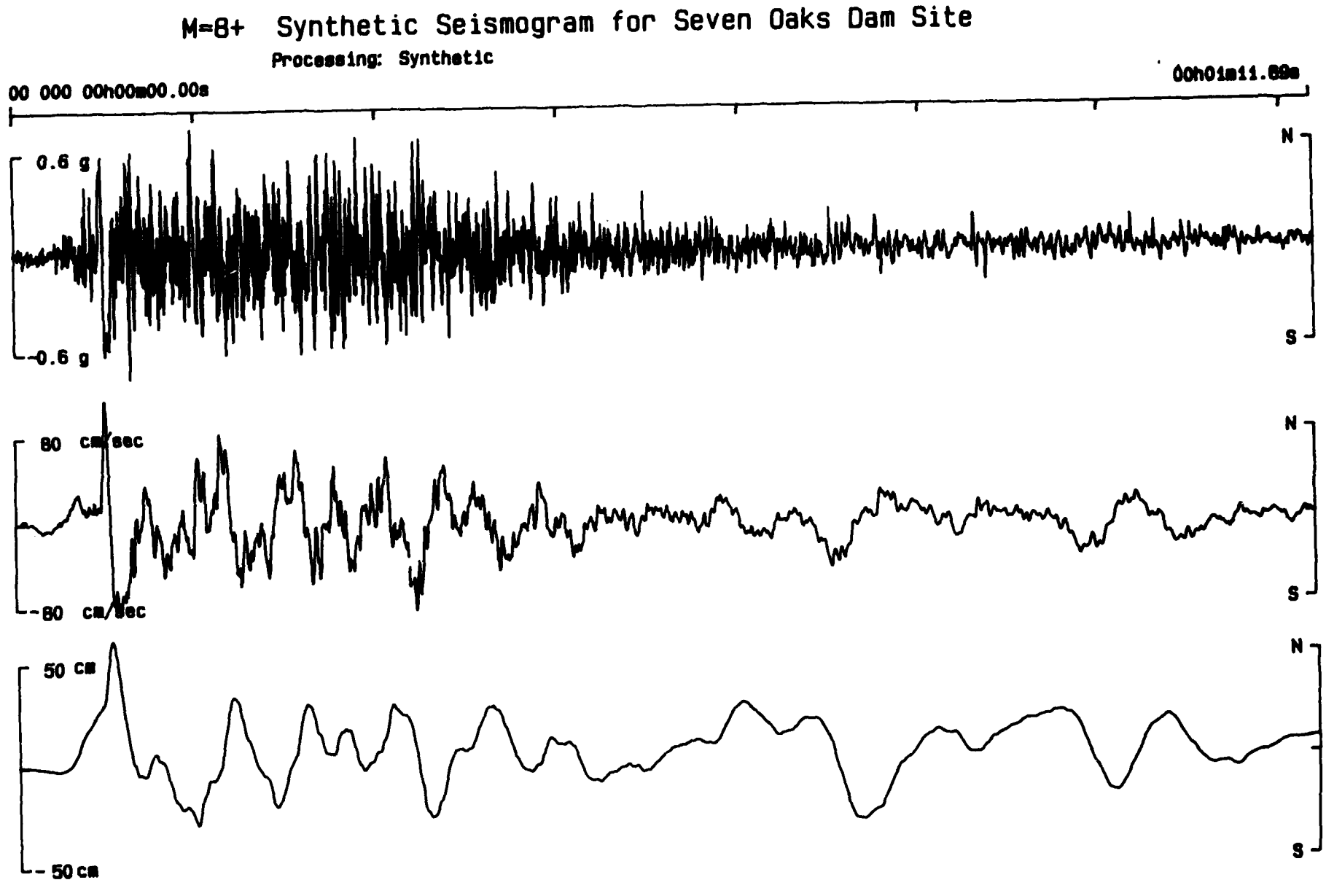
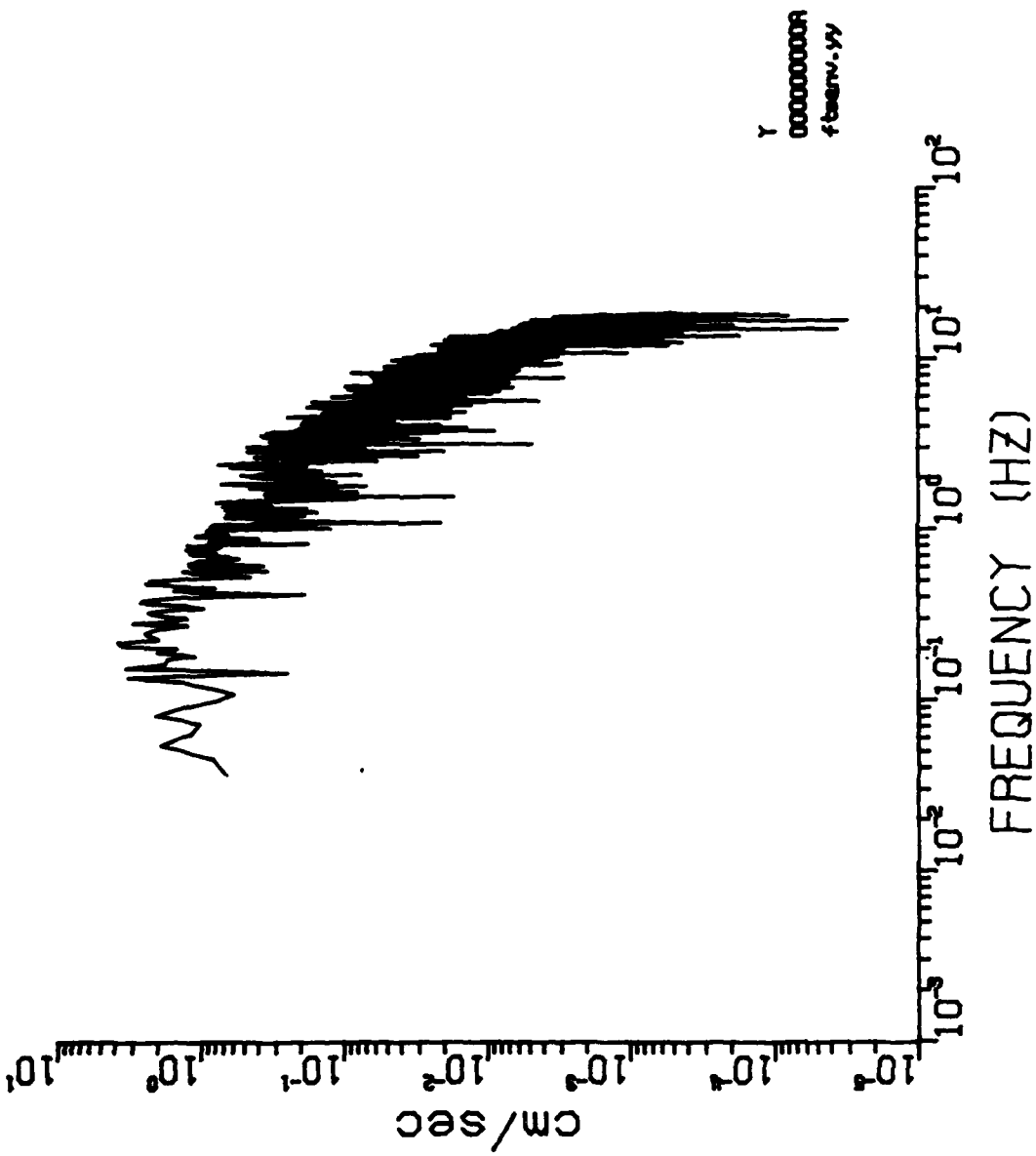


FIGURE 1 Time-histories: ground acceleration, velocity and displacement.

spectrum shown in Figure 2 shows a shape characteristic of many ground motion recorded in the near field of moderate earthquakes. There are no significant fluctuations or peaks or troughs in the velocity amplitude spectrum. Secondly, the normalized response spectrum (5 percent damping) for the recommended time history falls near the upper envelope of spectral shapes for similar curves from accelerograms at Taft, Koyna Dam and San Fernando (rock sites) and Imperial Valley (deep alluvium) - all smaller magnitude earthquakes. It is comparable with the spectral shape of the synthetic seismogram called Cal. Tech. A1.

The digital data for the accelerogram in Figure 1 are provided on a floppy disk. The peak digital value on the disk is normalized to unity (0.997) and requires scaling to 0.70 for input application in this case.

FIGURE 2 Undamped Fourier velocity spectrum for M=8+ synthetic seismogram.



VI. Effect of Nearby Significant Displacements

I am in general agreement with the definition and analysis of the probable occurrence and effects of significant fault displacements near to the dam, as set out in the Phase I Assessment (1986). In his consideration of the range of displacements that are credible on various components of the San Andreas fault system in the vicinity of the Seven Oaks Dam site, Professor Clarence Allen (Letter 12/8/1984) recommends the adoption of the assumptions that "as much as 4 feet of surficial fault displacement in any direction could take place, arbitrarily beneath the facility". The estimation of such displacements comes from field observations after previous large earthquakes associated with surface fault slip and from geomorphological and stratigraphic measurements along active faults (e.g. Bonilla, 1970; Sieh, 1978). A geotechnical case history of relevance in the present consideration is that of the proposed Auburn Dam the Sierra Nevada foothills in Northern California. Several geotechnical groups undertook extensive field work and evaluations of possible fault slip (e.g Idriss et al, 1977) that might occur near the site of the Auburn Dam. I was involved in the review of these studies.

My own evaluation would be that in the absence of any major active faults under the dam foundation or abutments it is perhaps more appropriate to consider the strain effects at the dam of 4 feet of displacement (or more) located on the nearby San Andreas fault planes. These effects would most likely be differential deformations and slips throughout the crustal volume adjacent to the fault, including shears along zones of weakness at the site.

Displacements would be carried from depth in the brittle rock up through the weaker (weathered) surficial materials. Variations in the strength and continuity of surface rock formations causing the strain dispersal by the shearing and cracking of rocks and finite slips over many joints and planes are, of course, unknown in detail so that prediction must be on the conservative side.

It might be expected, as pointed out in the Phase I Assessment (p. 8), that both horizontal and vertical displacements of the rock foundations under the dam could be accommodated by a multitude of adjustments within an earth-rock fill dam because of its low continuous rigidity and ability to deform differentially.

The measures for the design of a roller compacted concrete dam outlined in the Phase I Assessment (p. 19-21) to mitigate the effects of local elastic strain release indicate that engineering response to the problem is not completely infeasible. Because my own training is in applied mechanics and seismology rather than engineering I confine my response for comments on this design question largely to the general discussion above. With respect, however, to the models of deformation shown in Plate 6 of the Phase I Assessment, I stress again that, for the Seven Oaks damsite, the hazard is not likely to be a single major fault slip under the dam, as shown in the "worst case" sketches, but rather tensions and compressions due to the integrated displacements across the structure. Thus the close spacing of transverse vertical joints across the dam axis, designed to accommodate up to 4 feet of movement in any direction, overall might provide adequate response. The detail of faulting in the exposed foundation rock after the excavation would be the crucial factor in further evaluation of this question.

References

Abrahamson, N.A. and B.A. Bolt, "The Spatial Variation of the Phasing of Seismic Ground Motion", Bull. Seim. Soc. Am., 75, 1247-1264, 1985.

Bertero, V.V., S.A. Mahin, and R.A. Herrera, "Aseismic Design Implications of Near-Fault San Fernando Records", Earthquake Eng. and Structural Dyn. J., 6, 31-42, 1978.

Bolt, B.A., "Duration of Strong Ground Motion", Proc. Fifth World Conference on Earthquake Engineering, Paper 292, Rome, 1973.

Bolt, B.A., "The Interpretation of Strong Ground Motion Records", Misc. Report, 17, U.S. Army Engineers Waterways Experiment Station, S-73-1, 1-215, 1981

Bolt, B.A., "The Contribution of Directivity Focusing to Earthquake Intensities", Misc. Report, U.S. Army Engineers Waterways Experiment Station, 1983.

Bolt, B.A. and N.A. Abrahamson, "New Attenuation Relation for Peak and Expected Accelerations of Strong Ground Motion", Bull. Seism. Soc. Am., 72A, 2307-2322, 1982.

Bonilla, M.G., "Surface Faulting and Related Effects", in Earthquake Engineering, Prentice Hall, 1970.

Boore, D.N., W.B. Joyner, A.A. Oliver III and R.A. Page, "Peak Acceleration , Velocity, and Displacement from Strong Motion Records", Bull. Seism. Soc. Am., 70, 305-321.

Campbell, K.W., "Near-source Attenuation of Peak Horizontal Acceleration", Bull. Seism. Soc. Am., 71, 2039-2070, 1981.

Hanks, T.C. and R.K. McGuire, "The Character of High Frequency Strong Ground Motion", Bull. Seism. Soc. Am., 21, 2071-2095, 1981.

Idriss, I.M., M.S. Power, K. Sadigh, and R. Kulkarni, "Earthquake Ground Motions", in Earthquake Evaluation Studies of the Auburn Dam Area, U.S. Bureau of Reclamation, 1977.

Jennings, C.W., Fault Map of California, with locations of volcanoes, thermal springs and thermal wells: California Division of Mines and Geology, Geologic Data Map Series, n. 1, scale 1: 750, 000.1975.

Joyner, W.B. and D.M. Boore, "Peak Horizontal Acceleration and Velocity from Strong Motion Records Including Records from the 1979 Imperial Valley, California, Earthquake", Bull. Seism. Soc. Am., 71, 2011-2038, 1981.

Krinitsky, E.L. and F.K. Chang, "State-of-the-Art for Assessing Earthquake Hazards in the United States, Specifying Peak Motions for Design Earthquakes", Miscellaneous Paper S-73-1, Report 7, U.S. Army Engineer Waterways Experiment Station, Vicksburg, 1975.

Lindh, A.G., "A Preliminary Assessment of Long-Term Probabilities for Large Earthquakes along Selected Fault Segments of the San Andreas Fault System", U.S. Geological Survey Open-File Report, 63, 1-15, 1983.

Schnabel, P.B. and H. B. Seed, "Accelerations in Rock for Earthquakes in the Western United States", Bull. Seism. Soc. Am., 63, 501, 1973.

Seed, H.B. and I.M. Idriss, "Ground Motions and Soil Liquifaction during Earthquakes", E.E.R.I., Berkeley, 1982.

Sherard, J.L., L.S. Cluff and C.R. Allen, "Potentially Active Faults in Dam Foundations", Geotechnique, 24, 367-428, 1974.

Sieh, K., "Prehistoric large earthquake produced by slip on the San Andreas fault at Pallett Creek, California", Geophys, Res., 83, 3907-3939, 1978.

Trifunac, M.D. and A.G. Brady, "Correlation of Peak Acceleration, Velocity and Displacement with Earthquake Magnitude, Distance and Site Conditions", Earthquake Eng. and Structural Dyn. J., 4, 5, 1976.

Tsai, Y.B. and B.A. Bolt, "An Analysis of Horizontal Peak Ground Acceleration and Velocity from SMART 1 Array Data", Bull. Inst. Earth Sci., Academia Sinica, 3, 105-126, 1983

Weldon, R.J. and K. Sieh, "Holocene rate of slip and tentative recurrence interval for large earthquakes on the San Andreas fault in Cajon Pass, Southern California", Geological Soc. of Am. Bulletin, 96, 793-812, 1985.

Ziony, J.I. (Editor), "Evaluating Earthquake Hazards in the Los Angeles Region - An Earth - Science Perspective", U.S. Geological Survey Professional Paper, 1360, 1985.

A GEOTECHNICAL

ATTACHMENT A-3

Woodward-Clyde Consultants, Logs of Borings
and Reports on Aquifer Test and Geophysical Investigations

203 North Golden Circle Drive
Santa Ana, CA 92705
(714) 835-6886
(213) 581-7164
Telex 68-3420

Woodward-Clyde Consultants

7 December 1984
Project No. 41745C

Mr. C. Byron Rathburn
U.S. Army Corps of Engineers
300 North Los Angeles Street
Los Angeles, California 90012

SUBJECT: AQUIFER TEST
Upper Santa Ana River ~~MENTONE~~ DAM
SAN BERNARDINO COUNTY, CALIFORNIA

Dear Mr. Rathburn:

This letter presents the results of Woodward-Clyde Consultants aquifer test at the site of the proposed ~~Mentone~~ Upper Santa Ana River Dam in San Bernardino County, California. An aquifer test was performed during the period 7 November through 10 November 1984 at the site using a 10-inch diameter pumping well and six observation wells ranging from 15 to 428 feet radial distance from the puming well. The pumping test included a rest period prior to pumping to establish static water levels in the vicinity of the pumping well, a pumping period during which the pumping well was operated at a rate of 870 gallons per minute, average, and a recovery period following pumping during which the recovery of water levels in all wells were measured. Water levels were measured in all wells during the entire test by Woodward-Clyde engineers and geologists under the supervision of a Certified Engineering Geologist. This letter report summarizes the results of analysis of data obtained during that test.

The aquifer is the saturated alluvium in this portion of the Santa Ana River bed. The river is in a youthful valley, with steep bedrock sides. Static, non-pumping water level depths in the vicinity of the pumping well are approximately 12 feet, and depth to the base of the alluvium is approximately 75 to 85 feet in this vicinity. The aquifer thickness varies from about 50 to 70 feet and averages about 65 feet in the vicinity of the aquifer test wells. The alluvium consists of medium to coarse sand and gravel with abundant cobbles and boulders. The river occurs at about 150 feet from the pumping well and at the time of the test the river was flowing. The granite bedrock at the sides of the alluvial channel act as impermeable boundaries to the unconfined alluvial aquifer.

The impermeable boundaries bordering the aquifer in the vicinity of the pumping test showed up clearly in the test data as shown in the attached field calculation and plotting sheets. The test data (well readings and time) are also attached. The rate of change of drawdown versus the logrithm of time since pumping began, and recovery versus the logrithm of time since pumping ceased, showed an initial slope representative of the aquifer before boundary effects, an intermediate slope representative of the effects of the nearest boundary, and a later-time slope representative of the impact of boundaries from both sides of the channel. The image-well method was used to separate the effects of boundary conditions from the time-drawdown characteristics of the aquifer itself. In addition, a distance-drawdown analysis was performed which helped to distinguish the aquifer characteristics from the impact of the boundaries.

The average transmissivity of the aquifer was calculated to be 133,000 gallons per day per foot (gpd/ft). The aquifer exhibited a storativity of 0.015. Although the data indicate some recharge from the river was detected in the monitor well closest to the river, the data also indicate that the effects or recharge on the aquifer test were minimal, and were insignificant in comparison with the effects of aquifer boundaries. Based upon the range of aquifer thickness, the hydraulic conductivity, or permeability, of the aquifer ranges from about 1,900 gpd/sq ft to 2,700 gpd/sq ft. The average permeability is about 2,200 gpd/sq ft, or about 0.10 cm/sec. The storativity value of 0.015 is in the range expected for a water table aquifer, but is approximately an order of magnitude lower than the estimated specific yield of the material. This difference is characteristic of a layered aquifer with horizontal layers varying from fine to coarse material. Based on the depositional environment of the alluvium, the storativity value of 0.015 is reasonable and is considered representative of the aquifer in this area.

The following summarizes the aquifer characteristics based on the data available:

Aquifer Material	Coarse Alluvium
Thickness:	
Range	50 to 70 feet
Average	65 feet

Transmissivity, average	133,000 gpd/ft
Storativity	0.015
Hydraulic Conductivity, or Permeability:	
Range	1,900 to 2,700 gpd/sq ft
Average	2,200 gpd/sq ft
(0.10 cm/sec)	

We trust this information satisfies your needs. If you have any questions, please call.

Very truly yours,

WOODWARD-CLYDE CONSULTANTS

Allen M. Yourman Jr.

Allen M. Yourman, Jr.
Project Engineer

AMY/ea
Enclosure

203 North Golden Circle Drive
Santa Ana, CA 92705
(714) 835-6886
(213) 581-7164
Telex 68-3420

Woodward-Clyde Consultants

14 January 1985
Project No. 41745C

Mr. C. Byran Rathburn
U.S. Army Corps of Engineers
300 North Los Angeles Street
Los Angeles, California 90012

SUBJECT: GEOPHYSICAL INVESTIGATION
UPPER SANTA ANA CANYON
SAN BERNARDINO, CALIFORNIA

Dear Mr. Rathburn:

1.0 GEOPHYSICAL MEASUREMENTS

1.1 Introduction

This letter presents Woodward-Clyde Consultants' (WCC) seismic crosshole and downhole measurements at the upper Santa Ana Canyon Dam site, San Bernardino County, California during November and December 1984. The purpose of the geophysical survey was to evaluate compressional and shear wave velocities in the subsurface soils.

The scope of testing was determined by the Los Angeles District, Corps of Engineers. The measurement program consisted of three downhole tests and three crosshole tests each at five-foot intervals. The locations of the test holes are shown in Figure 1.

This report describes the field operations and presents the compressional and shear wave velocities as interpreted from the field records.

1.2 Field Operations

1.2.1 Crosshole Measurements

Crosshole shear wave velocity measurements were obtained in three sets of three borings: Borings 1, 2, and 3 (Figure 2); Borings 5, 6, and 7 (Figure 3); Borings 11, 12, and 13 (Figure 4). Borings were cased with 3-inch ID PVC pipe and the annuli backfilled to provide acoustic coupling between the shear wave hammer, geophones, and surrounding soils. The hammer was placed in an end boring within each set of borings, and travel-time measurements were made at five-foot intervals to the depth of the shallowest boring within each set.

The basic field technique consists of measuring the time required for a shear (S) wave to travel from a borehole source to one or more receivers in other boreholes. The source and receivers are located at the same elevation at known distances from each other. The velocity of the shear wave (Vs) is calculated from the time-distance relationships.

The instrumetation used in the crosshole seismic survey included a reversible shear-wave hammer, two 4.5-Hz downhole geophones, and a Geometrics ES-1210F signal-enhancement seismograph. The hammer and geophones are clamped to the borehole walls pnuematically. A Terraprobe inclinometer was used to evaluate borehole deviation.

1.2.2 Downhole Measurements

Downhole shear wave and compressional wave velocity measurements were made in Borings 2, 5, and 6 (Figures 5, 6, and 7, respectively). Measurements were made at five-foot intervals from the surface to the bottom of each hole.

The basic field operation consists of measuring the time needed for the compressional (P) and shear (S) waves to travel from a surface source to a receiver at a known depth in the borehole.

The instrumentation used in the downhole seismic surveys included a hammer source, a triaxial downhole geophone, and a Geometrics ES-1210F signal-enhancement seismograph. Compressional waves are generated by a hammer blow to the ground. Shear waves were generated by reversing hammer blows to a vane-type shear wave bar embedded at the surface.

1.2 Results

The velocity and time-distance data are presented in Figures 2 to 7 as indicated above. Two velocity zones may be identified on the basis of these data: surficial low velocity soils about twenty feet thick, and moderate velocity river gravels to the base of the borings.

The surficial soils shear wave velocities average 750 fps (feet per second) and 770 fps respectively for the downhole and crosshole data. Compressional wave velocities average 1520 fps for the shallow soils downhole data. Compressional wave velocities increase downward to an average of 7500 fps due to water saturation of the river gravels. Shear wave velocities in the river gravels average about 2000 fps and 1600 fps respectively for the downhole and crosshole data.

The difference in the downhole and crosshole shear wave velocities may be attributed to the refraction of seismic waves along the highest velocity pathways, resulting in artificially high values in the downhole measurement values. Lateral and vertical variations in gravel and boulder content and point-to-point contact cause localized velocity variations, data scatter, and poor to fair data quality. No inclinometer measurements could be obtained in the Borings 5, 6, and 7 data set (due to storm damage), however the values are internally consistent and interhole inclination variations were insignificant in the other deep borings.

We trust this letter provides the information you require at this time. If you have any questions, please call.

Very truly yours,

WOODWARD-CLYDE CONSULTANTS

Allen M. Yourman
Project Engineer
RCE 32364

Ronald L. Mees
Project Geophysicist
GP 882

AMY:RLM/ea
Enclosures

BASED ON UPPER SANTA ANA RIVER DAM, 125,000 AC. FT. STORAGE. TOE 1500 FT. UPSTREAM OF POWER-HOUSE ROAD, DACW 09-84-C-0001, SHEET 8 OF 14.

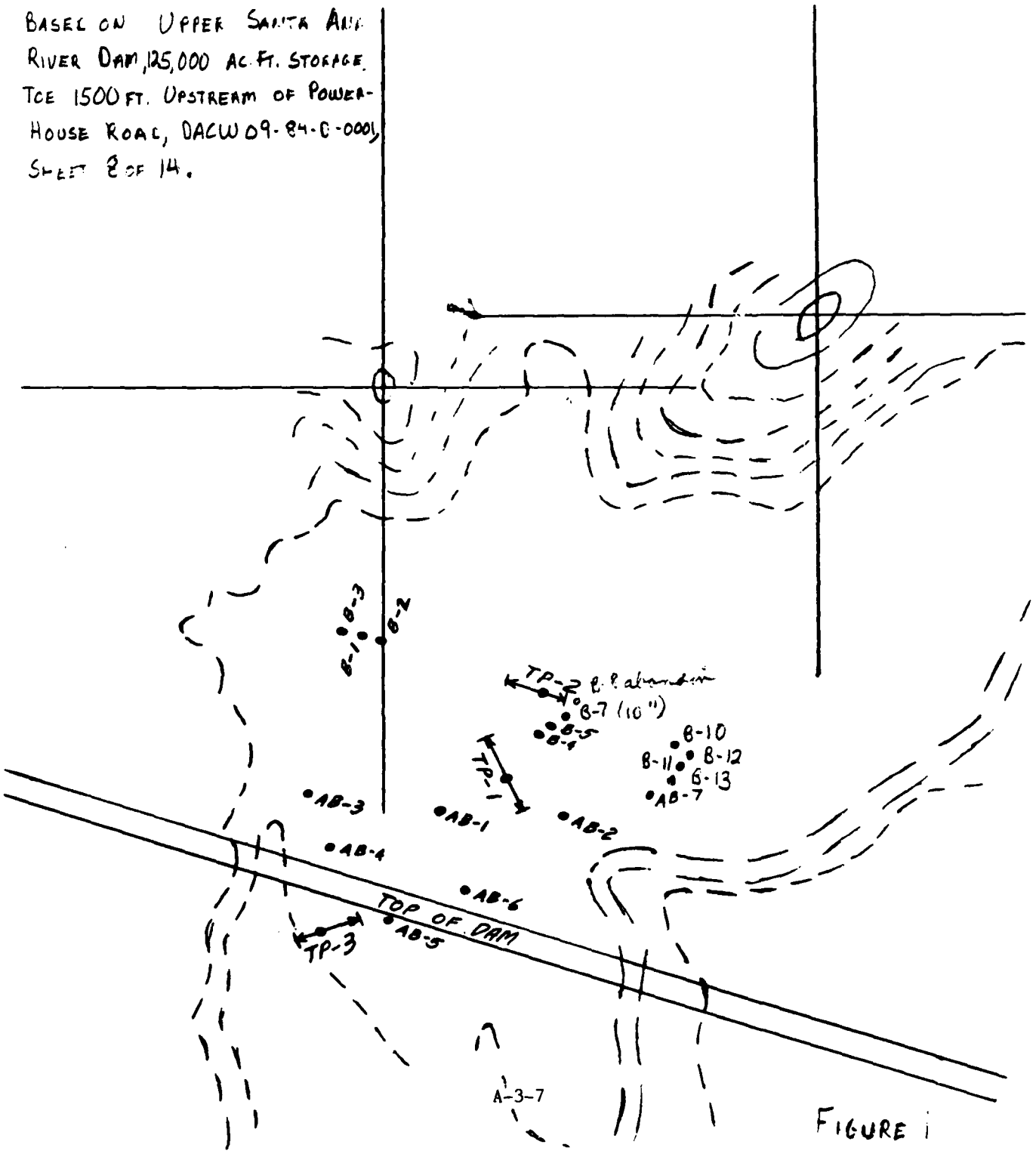


FIGURE 1

Woodward-Clyde Consultants

Project Upper Santa Ana Canyon Dam
Phase Crosshole Seismic Data
Subject Borings 1, 2, 3
Calculation No.: WCC

Sheet No. 1 of 3
File
Made by RCM Date 12-28-84
Checked by Date

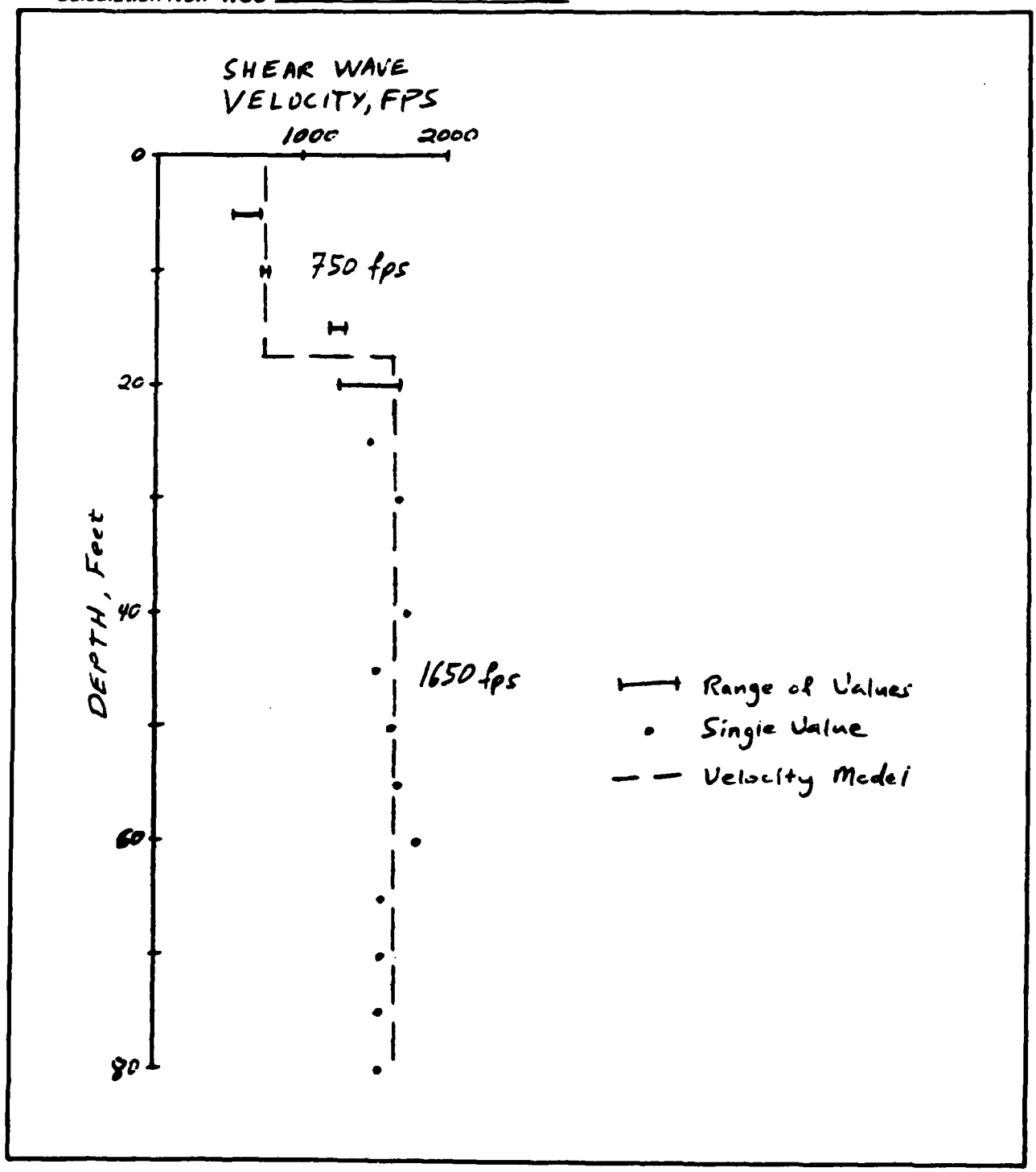


FIGURE 2

Woodward-Clyde Consultants

Project Upper Santa Ana Canyon Dam
Phase Cross-hole Seismic Data
Subject Borings 5, 6, 7
Calculation No.: WCC

Sheet No. 2 of 3
File
Made by RCM Date 12-28-84
Checked by Date

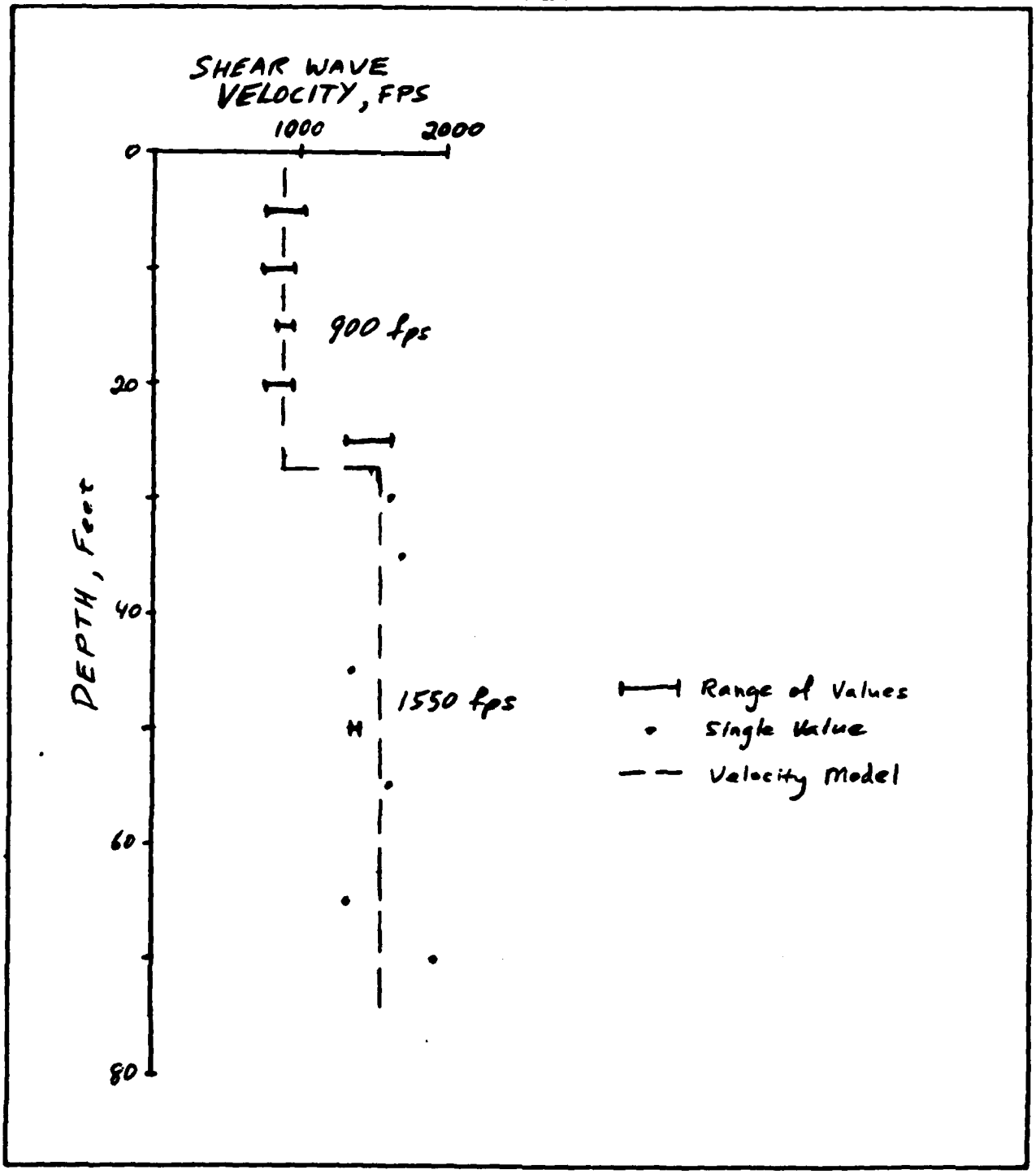


FIGURE 3

Woodward-Clyde Consultants

Project Upper Santa Ana Canyon Dam

Phase Crosshole Seismic Data

Subject Borings 11, 12, 13

Calculation No.: WCC

Sheet No. 3 of 3

File

Made by RLM Date 12-28-84

Checked by Date

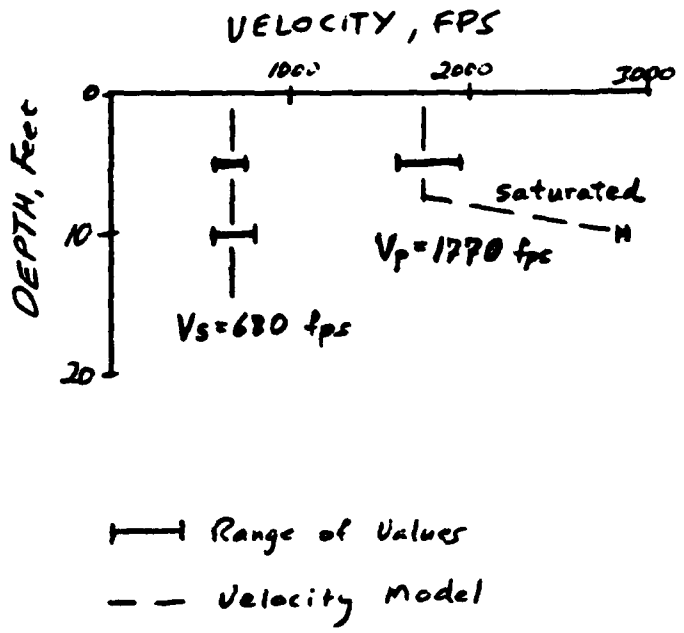


FIGURE 4

Woodward-Clyde Consultants

Project Upper Santa Ana Canyon Dam
Phase Downhole Seismic Data
Subject Boring 2
Calculation No.: WCC

Sheet No. 1 of 3
File
Made by [illegible] Date 12-25-84
Checked by Date

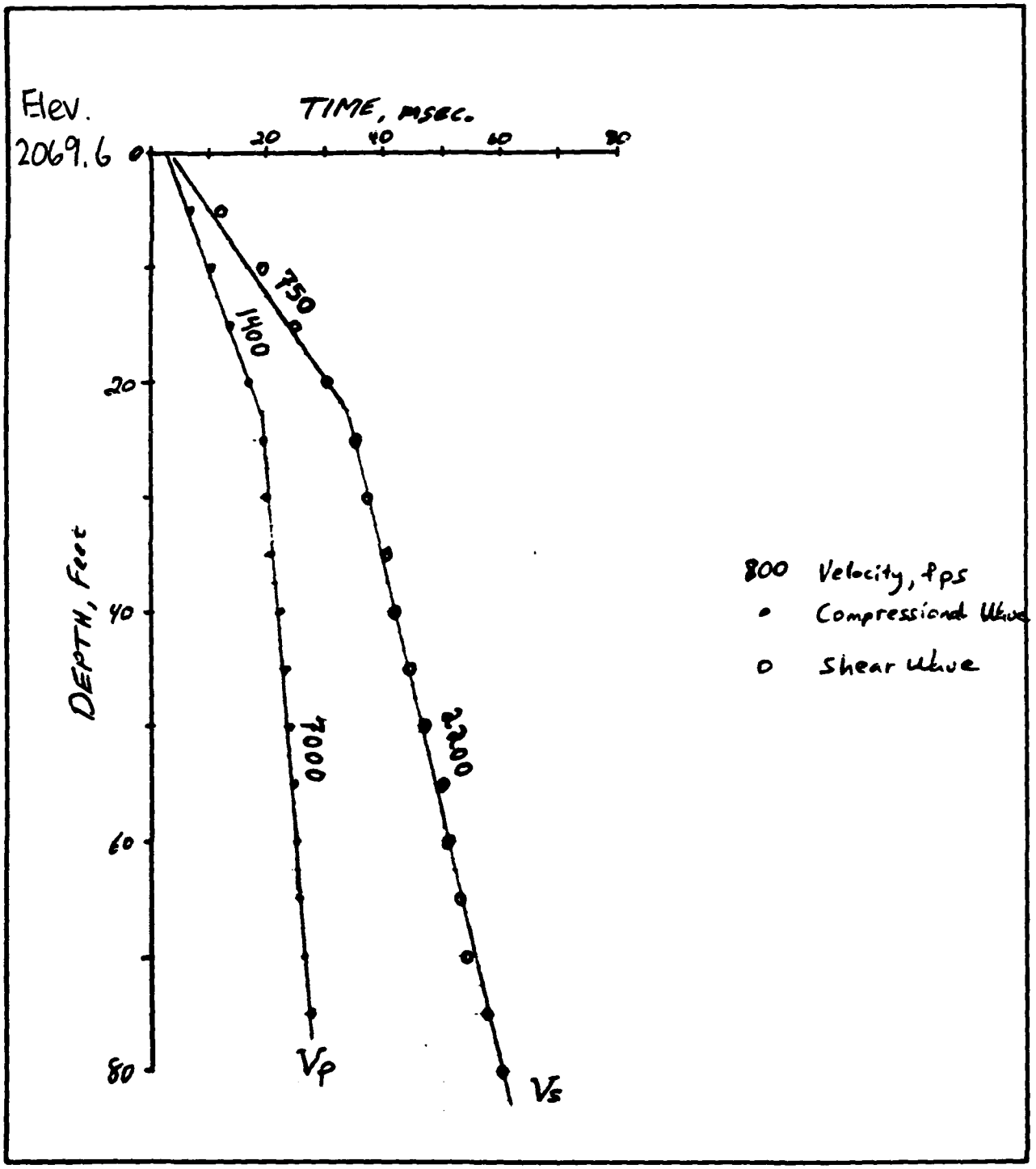


FIGURE 5

Woodward-Clyde Consultants

Project Upper Santa Ana Canyon Dam

Phase Downhole Seismic Data

Subject Boring 5

Calculation No.: WCC

Sheet No. 2 of 3

File

Made by RLM Date 12-25-84

Checked by Date

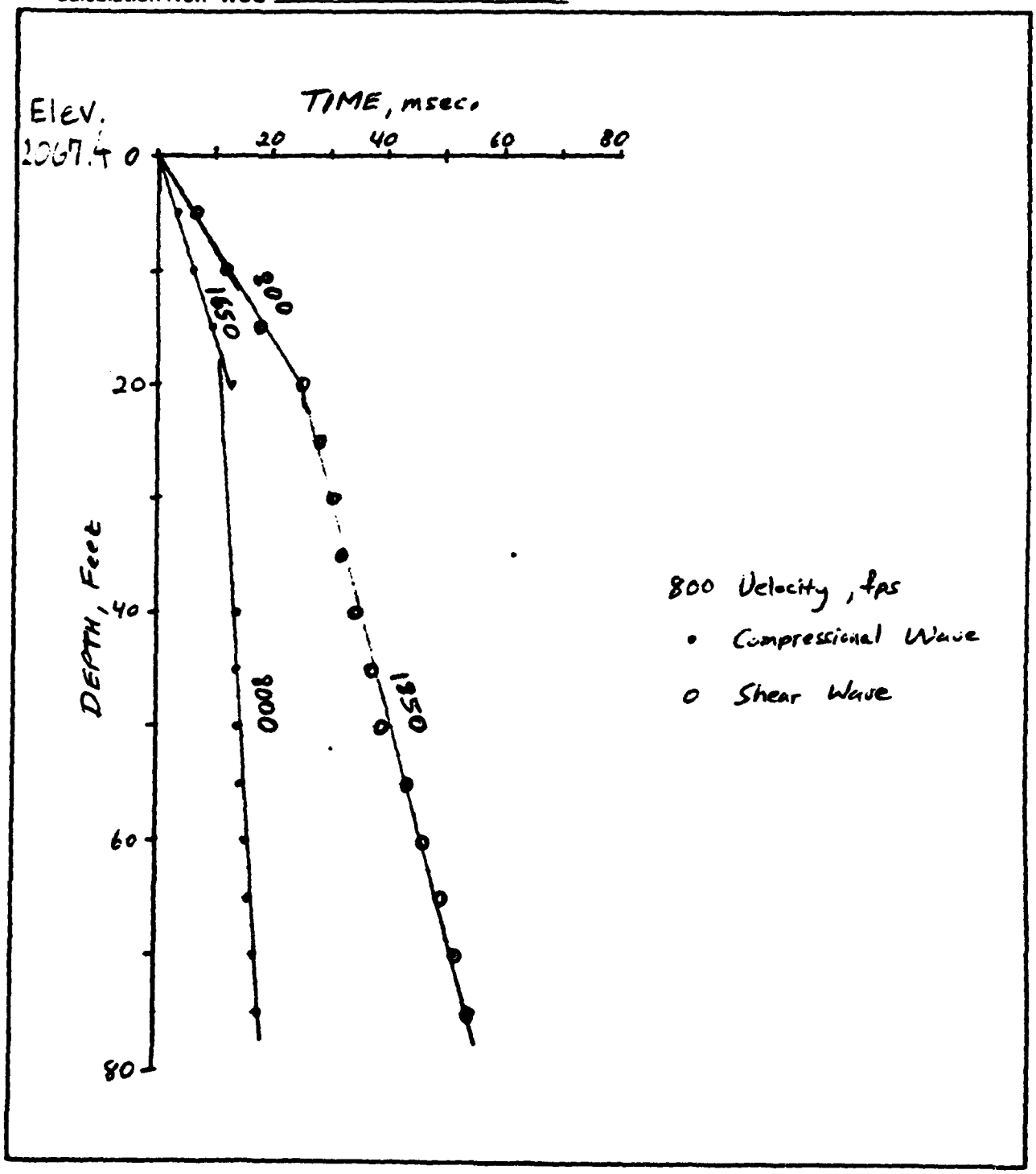


FIGURE 6

Woodward-Clyde Consultants

Project Upper Santa Ana Canyon Dam
Phase Downhole Seismic Data
Subject Boring 6
Calculation No.: WCC

Sheet No. 3 of 3
File
Made by RM Date 12-28-84
Checked by Date

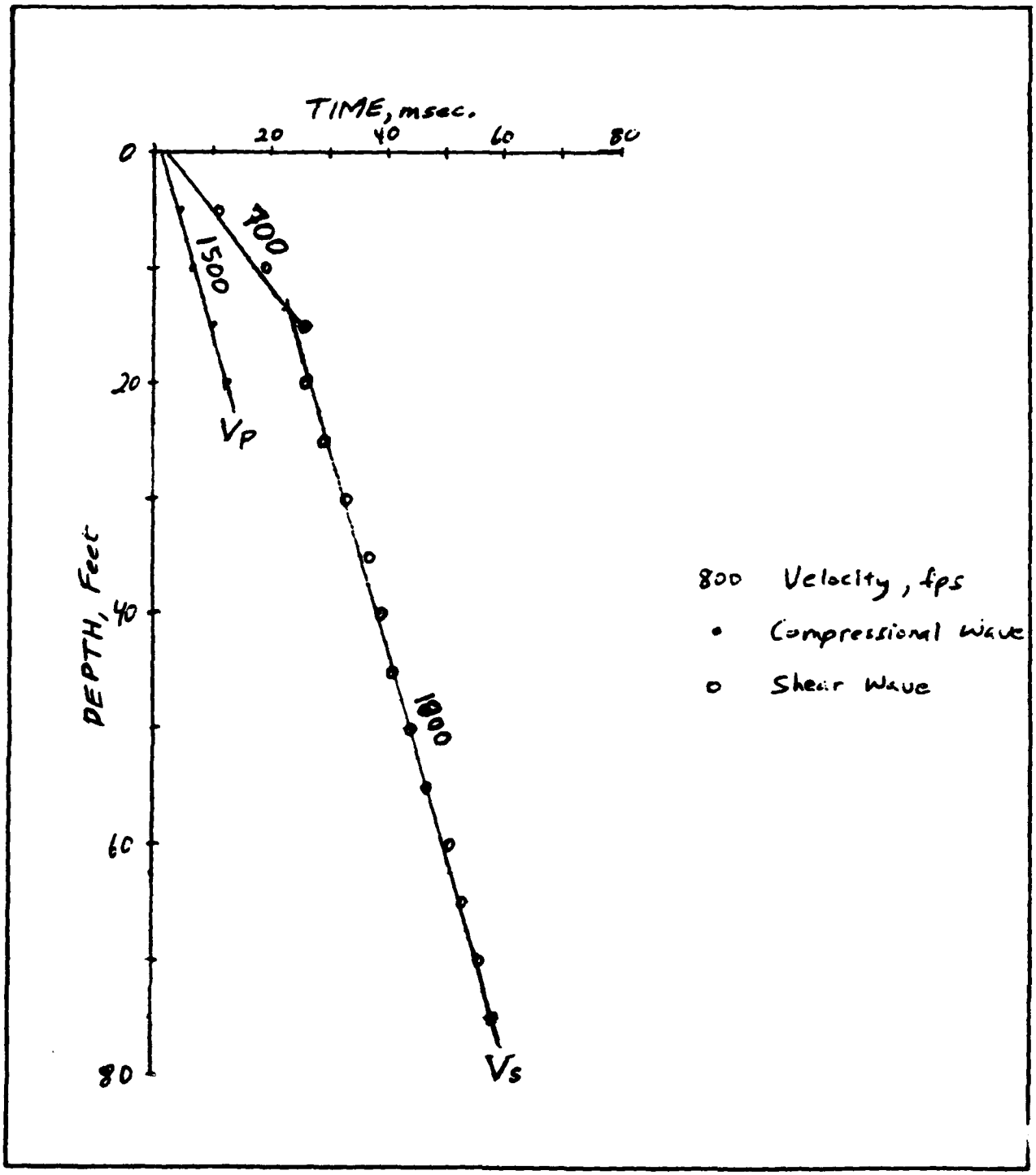


FIGURE 7

203 North Golden Circle Drive
Santa Ana, California 92705
(714) 835-6886 (213) 581-7164
Telex 68-3420

Woodward-Clyde Consultants

LETTER OF TRANSMITTAL

TO: U.S. Army Corps of Engineers DATE: 8 January 1985

300 North Los Angeles Street PROJECT NO: 41745C

Los Angeles, California 90012

ATTENTION: C. Byran Rathburn

SUBJECT PROJECT: ~~Mentone~~ Upper Santa Ana River Dam

IN ANSWER TO YOUR REQUEST OF: --

WE ARE TRANSMITTING: (X) HEREWITH UNDER SEPARATE COVER

THE FOLLOWING: Copies of the logs of test borings and site plan and keys to access gates.

FOR YOUR:

APPROVAL	FILES
COMMENTS	INFORMATION
DISTRIBUTION	(X) USE

REMARKS: The boring locations were estimated based upon angles taken from on-site electrical towers.

PLEASE NOTIFY US IF ENCLOSURES LISTED ARE NOT RECEIVED.

COPIES TO:

Very truly yours,
WOODWARD-CLYDE CONSULTANTS

By Allen M Yourman Jr.
Allen M. Yourman, Jr.

DATE OF BORING 10-5-84 WATER DEPTH 28.5ft DATE MEASURED 10-5-84
TYPE OF DRILL RIG Becker Reverse Circulation HOLE DIAMETER 7 ins
WEIGHT OF HAMMER 4000 Ft lbs/in² FALLING _____ SAMPLES none

DEPTH, FT.	SAMPLES	BLOWS/FOOT	DESCRIPTION	UNC. COMP. STRENGTH, ksf	MOISTURE CONTENT, %	DRY DENSITY pcf	OTHER TESTS
			Start Drill 0924 hrs 10/5/84				
			SURFACE ELEVATION:				
			Loose Damp, Sand (SP), fine to coarse, fine to coarse gravel and cobbles to 5 inches. 76-20-5				
			Med Dense, Dry brown				
5			Dense				
	100/6"		Very Dense, Boulder 18"				
10	24		Med Dense Damp Sand (SP), fine to medium, gravel to 2ins 20%				
	24						
	26						
	27						
	28						
15	89						
	73						
	56						
	146/9"		Moist 17.75 ft.				
20			Boulder 17.75 to 21.50 = 3.75 ft.				
	100/6"		Moist, Dense, Light Brown, Sand (SP), fine to coarse, fine Gravel				
	90						
	94		coarse, fine to coarse gravel 30%				
25	91						
	87		fine to med, f to c gravel 20%				
	64						
	57		fine to coarse, fine to coarse Gr 20%		Water at 28.5 ft		
	36	▼					
	66		Wet Lt Brown, Sandy Gravel (GP) fine to coarse, occasional cobbles.				
30	94						
	74		Wet Lt Brown Gravelly Sand (SP) fine to coarse 80% sand 20% Gr.				
	60						
	64						
35	66						
	86						
	70						
	51		170/6" to 38.5'				
			Wet, Lt Brown Sandy Gravel, fine to coarse, occasional Cobbles				
40	320						
	62		Wet, Lt Brown Gravelly Sand, coarse, occasional cobbles (SP)	GR 25	SD 75 %		
	48						
	58						
45	54						
	86						
	150						
	152						
	160						
	149						
50	125						

Project: Mentone Dam.
Project No. 41745C

LOG OF BORING NO 1

Fig.

LA/OR-0181-284

WOODWARD - CLYDE CONSULTANTS

DATE OF BORING 10-5-84 WATER DEPTH 266.5ft DATE MEASURED 10-3-84
TYPE OF DRILL RIG Becker Reverse Circulation HOLE DIAMETER 7 ins
WEIGHT OF HAMMER 8000ft lbs/in² FALLING _____ SAMPLES none.

DEPTH, FT.	SAMPLES	BLOWS/FOOT	DESCRIPTION	UNC. COMP. STRENGTH, tsf	MOISTURE CONTENT, %	DRY DENSITY pcf	OTHER TESTS
			SURFACE ELEVATION:				
55	87 91 86 90 86 213 171 119 91		Wet, Lt Brown, Gravelly Sand (SP) coarse sand occ cobbles. 80% Sand, 20% Gravel fine to coarse Gr, S Gr C 70 20 10 more cobbles				
60	81 70 50 107 145						
65	14.3 min/ft (93 blows/min). Gravelly Sand (SP), wet to coarse sand fine to coarse gravel occ cobbles. .7 min/3" Wet, Lt Brn, Gravelly Sand (SP) fine to coarse gravel fine, no cobbles.						
70			Stopped drilling at 1430hrs @ 65.25ft. Blow count at 64-65ft 1349 blows/ft Blow count at 65-65.25ft 1580 blows/3ins				
75							
80							
85							
90							
95							
100							

Project: Mentone Dam
Project No. 41745C

LOG OF BORING NO 1

Fig.

DATE OF BORING 10 October 1984 WATER DEPTH 29 ft DATE MEASURED 10-10-84
TYPE OF DRILL RIG Becker Reverse Circulation HOLE DIAMETER 7 ins
WEIGHT OF HAMMER 8000 ft lbs/in² FALLING _____ SAMPLES _____

DEPTH, FT.	SAMPLES	BLOWS/FOOT	DESCRIPTION	UNC. COMP. STRENGTH, tsf	MOISTURE CONTENT, %	DRY DENSITY pcf	OTHER TESTS
			SURFACE ELEVATION:				
	2		Damp, Brown, Sand (SP) fine to coarse grain				
	1						
	Push						
	Push						
5	6		Damp, Lt Brn, Sand (SP) F to C grain, Gravel fine to coarse				
	28						
	60						
	60		Damp, Lt Gray Sand (SP) F to C grain, Gravel fine to coarse occasional cobbles				
	61						
10	47						
	24						
	25						
	43						
	104						
	83						
15	44		Damp, Lt Brn, Sandy Gravel (GP) coarse grain, fine to coarse gravel, cobbles with angular side				
	71						
	93						
	100						
20	46						
	48						
	66						
	76						
	60						
25	94						
	93						
	94						
	114						
	76	▼	Water @ 29 ft Saturated. Free				
	48		Saturated, Lt Gray-Brown, Gravelly Sand (SP) med to coarse grain, fine to coarse gravel, cobbles with angular sides Boulder at 30 ft to 31 ft.				
30	159						
	56						
	64						
	61						
35	71						
	77						
	70						
	104						
	72						
40	26						
	21						
	76		Boulder 41 ft to 46 ft.				
	370	B					
	767						
45	115						
	150						
	84		Lt Brn, Gravelly Sand (SP) med to coarse grain, fine to coarse gravel, cobbles w/ angular side.				
	91						
	126						
50	68						

Project: Mentone Dam
Project No. 41745C

LOG OF BORING 2

Fig.

DATE OF BORING 10 October 1984 WATER DEPTH 29 feet DATE MEASURED 10-10-84
TYPE OF DRILL RIG Becker Reverse Circulation HOLE DIAMETER 7 ins
WEIGHT OF HAMMER 8000 ft lbs/in² FALLING _____ SAMPLES _____

DEPTH, FT.	SAMPLES	BLOWS/FOOT	DESCRIPTION	UNC. COMP. STRENGTH, ksf	MOISTURE CONTENT, %	DRY DENSITY pcf	OTHER TESTS
			SURFACE ELEVATION:				
50		40 41 62 45	Lt. Brown, Gravelly Sand (SP). med to coarse grain, fine to coarse gravel, occasional cobbles or boulders of angular side				
55	5	84 62 96 88	Lt Gray, Sandy Gravel (GP) coarse, cobbles or boulders of angular side				
60	10	61 77 1031 1993	Boulder 60'-63'. Use rotary drilling from 62ft to 63ft. Tricone.				
	10½	58					
65	15	234 174 273 76	Lt Yellow Brown, Gravelly Sand (SW) coarse grain, with fine to coarse gravel, some cobbles with angular side, gravel and cobbles of granitic nature.				
70	20	65 46 342 697	Lt Yellow Brown, Sandy Gravel (GP), fine to medium grain and fine gravel	drill	71ft	10-10-84	
		154	Boulder 70ft to 72.5ft	10-11-84 drilling 71ft to 95½ft			start
75	25	89 96 207	Lt Gray Brown, Sandy Gravel (GP), fine to coarse grain, fine to coarse gravel, cobbles or boulders with angular side				
80	30	186 129 74 78 64 72 62 88	Lt Gray Brown, Sandy Gravel (GP) coarse grain, fine to coarse gravel, cobbles or boulders with angular side, granitic in nature.				
85	35	394 212 308 171 139	Lt Gray Brown Sand (SP), fine to medium grain, fine gravel fine to coarse gravel angular on one side granitic in nature.				
90	40	36 136 80 166					
		320	@95.5ft cuttings gray Sandy Clay (CL) fine, probable top of bedrock.				
95	45	309 246/3"	Bottom of Hole 95.25ft.				
100							

Project: Mentone Dam
Project No. 41745C

LOG OF BORING 2

Fig.

DATE OF BORING 12 October 1984 WATER DEPTH _____ DATE MEASURED ~~10-12-84~~

TYPE OF DRILL RIG Becker Reverse Circulation HOLE DIAMETER 7 ins

WEIGHT OF HAMMER 4000 ft lbs/ln² FALLING _____ SAMPLES _____

DEPTH, FT.	SAMPLES	BLOWS/FOOT	DESCRIPTION	UNC. COMP. STRENGTH, ksf	MOISTURE CONTENT, %	DRY DENSITY pcf	OTHER TESTS
			Start drilling with new Falcon Bit.				
			Start Drilling 10-12-84 @ 1058 hrs.				
			SURFACE ELEVATION:				
0		3	Damp, Brown, Sand, (SP-SM), fine to medium grain.				
		Auger Refusal					
5		35	Damp, Lt Brown, Sand (SP), fine to coarse grain, fine and course				
		36	Gravel, cobbles and possible boulders with				
		77	Boulder angular side. granitic in nature.				
		61	or				
		136	Large Cobbles - no sand				
10		97	2inch Loose Sand (SP) fine to med grain.				
		120	Boulder granitic				
		6 Blows for 3 ins.	Change to Rotary & tricone 3-5 min for 3 ins.				
		15 Blows for 1 ft.					
14-5		14 Blows for 1 ft					
15			Bottom of Hole 14 ft. Time 1540 Hrs.				
			Driller changed back to Falcon Bit				
			and was unable to advance the hole				
			through the area of the rotary drilling				
20			Driller stopped drilling with Falcon				
			Bit at 11.75 ft.				
			Abandoned by Becker				
25							
30							
35							
40							
45							
50							

Project: Montana Dam
Project No. 41745C

LOG OF BORING 3

Fig.

DATE OF BORING 13 October 1964 WATER DEPTH _____ DATE MEASURED _____

TYPE OF DRILL RIG Becker Reverse Circulation HOLE DIAMETER 7 ins

WEIGHT OF HAMMER 3000 ft lbs/in² FALLING _____ SAMPLES _____

DEPTH, FT.	SAMPLES	BLOWS/FOOT	DESCRIPTION	UNC. COMP. STRENGTH, ksf	MOISTURE CONTENT, %	DRY DENSITY pcf	OTHER TESTS
			Start Drilling at 1605 Hrs				
			SURFACE ELEVATION:				
		Push	Damp, Lt Brown, Sand (SP-SM), fine to medium grain.				
		1					
		3					
		2.1	Damp, Lt Brown, Sand (SP), medium to coarse grain, fine to coarse gravel.				
5		115	Boulder Borehole about 12° from vertical at 8-5 feet				
		170	Pulled out the drill bit and bounce pipe start to				
		43	redrill near vertical. Stopped drilling because				
		20	location of hole does not meet Engineers location requirement.				
		14/6"	Stopped Drilling at 8.5 ft Bore holes should not be more than 15 ft apart				
10							
			Abandoned by Woodward-Clyde Consultants				
15			After straightening the hole stopped at 6 ft.				
20							
25							
30							
35							
40							
45							
50							

Project: Mentone Dam
Project No. 41745C

LOG OF BORING 4

Fig.

LA/OR-0181-284

WOODWARD - CLYDE CONSULTANTS

DATE OF BORING 13 October 1984 WATER DEPTH 29.5 ft DATE MEASURED 10-13-84

TYPE OF DRILL RIG Becker Reverse Circulation HOLE DIAMETER 7 ins

WEIGHT OF HAMMER 8000 ft lbs/in² FALLING _____ SAMPLES _____

DEPTH, FT.	SAMPLES	BLOWS/FOOT	DESCRIPTION	UNC. COMP. STRENGTH, tsf	MOISTURE CONTENT, %	DRY DENSITY pcf	OTHER TESTS
			Start drilling at 1104 Hrs.				
			SURFACE ELEVATION: 0				
		Push	Damp, Lt. Brown, Sand (SP), fine to medium grain.				
		7					
		12					
		15					
5		46	Damp Lt Brown sand (SP) fine grain, Gravel fine to coarse and				
		66	Cobbles or Boulders with angular side, granitic in nature.				
		50					
		34					
		96					
10		70	Boulder or Boulders, granitic in nature.				
		29		at 11ft hole out of vertical driller correcting and redrill to 11ft.			
		44					
		64					
		44					
15		36	Damp, Lt Brown, Sand (SP) fine to medium grain, Gravel fine to coarse				
		36					
		26					
		331	Cobbles or Boulder				
		264					
20		106					
		60	Moist, Brown, Sand (SP) med to coarse grain, Gravel, fine to coarse				
		110	w/ Cobbles or Boulders with angular side, granitic in nature				
		57					
		210	Cobble				
25		70	6inch lense Sand (SP), fine to medium grain				
		124					
		155	Cobbles and Sandy Gravel, medium to coarse grain, Gravel fine to coarse				
		73					
	▼	167	Lt Brown, Sandy Gravel, med to coarse grain, Gravel fine to Coarse				
30		27	w/ Cobbles, angular side granitic in nature				
		31					
		52					
		52					
		26					
35		71					
	WATER	600	Boulder at 35.5ft				
		2520					
		1651					
		744					
40		164	Lt Brown, Sandy Gravel (GP) Sand med to coarse grain, Gravel fine to				
		94	coarse, Cobbles with angular side, others rounded, granitic.				
		100					
		134					
		107					
45		104					
		98					
		27					
		52					
		70					
50		98					

Project: Martone Dam
Project No. 41745C

LOG OF BORING 5

Fig.

LA/OR-0181-204

WOODWARD - CLYDE CONSULTANTS

DATE OF BORING 13 & 15 October 84 WATER DEPTH 29.5 ft DATE MEASURED 10-13-84

TYPE OF DRILL RIG Becker Reverse Recirculation HOLE DIAMETER 7 ins

WEIGHT OF HAMMER 8000 ft lb/in² FALLING _____ SAMPLES _____

DEPTH, FT.	SAMPLES	BLOWS/FOOT	DESCRIPTION	UNC. COMP. STRENGTH, tsf	MOISTURE CONTENT, %	DRY DENSITY pcf	OTHER TESTS
			SURFACE ELEVATION:				
50	72		Lt Brown Sand (SP) fine to medium grain				
	77						
	94		Lt Brown, Sandy Gravel (GS), fine to coarse Sand, fine to Coarse gravel Cobbles or possible Boulders, angular on one side granitic in nature				
	66						
55	91						
	85						
	102						
	70						
	76						
60	63						
	120						
	250						
	145						
	150						
65	136						
	91						
	88						
	92						
	21		Lt Brown Sand (SP) fine to medium grain, fine gravel				
70	45		fine to coarse gravel				
	160		Lt Brown Sandy Gravel (GP) medium to coarse sand, fine to coarse gravel, and cobbles or boulders, angular on one side granitic in nature.				
	91						
	90						
	168						
75	414		Lt Brown, Sand (SP) fine to medium to coarse grain, fine Gravel.				
	260		Lt. Brown Sand (SP) fine to medium, fine to coarse gravel. Cobbles & Boulders from 74-5 ft. angular on one side granitic in nature.				
	664						
	298						
	102						
80	107		SW				
	86		Lt. Gray-Brown Gravelly Sand (SW), fine to coarse grain sand, fine to coarse gravel, cobbles or boulders, angular on one side granitic in nature.				
	115						
	75						
	33						
85	41						
	82		Lt Gray-Brown Sandy Gravel (GW) fine to coarse sand, fine to coarse Gravel, cobbles or boulders, angular one side, granitic				
	69						
	83			89 ft drill		10-13-84	
	130						
90	106				10-15-84		
	256		Lt Gray, Sand with weathered fine gravels, granitic, weathered easily crumbled, no cobbles. Possible contact with bedrock				
	262						
	89						
	87						
95	50						
			Bottom of Hole at 95 ft. Time 15.18 hrs.				
100	50						

Project: Mentone Dam

Project No. 41745C

LOG OF BORING 5

Fig.

DATE OF BORING 16 October 1984 WATER DEPTH 13.5 ft. DATE MEASURED 10-16-84
TYPE OF DRILL RIG Becker Reverse Circulation HOLE DIAMETER 7 ins
WEIGHT OF HAMMER 8000 ft lbs/in² FALLING _____ SAMPLES _____

DEPTH, FT.	SAMPLES	BLOWS/FOOT	DESCRIPTION	UNC. COMP STRENGTH, ksf	MOISTURE CONTENT, %	DRY DENSITY pcf	OTHER TESTS
			Start Drilling at 0925 Hrs.				
			SURFACE ELEVATION:				
		14	Loose Moist Brown, Sand (SP), fine to medium grain, fine and				
		101	Cobbles coarse gravel, and cobbles				
		156	or boulders, different colors of rock fragments and textures.				
		144	Light and Dark Gray, and Pink.				
5		144					
		101					
		69	Damp Dense, Lt Brown, Sandy Gravel (GP) med to coarse grain, fine to coarse gravel.				
		120	Cobbles or Boulder.				
		126					
10		90	Damp Dense, Lt Brown, Sandy Gravel (GP) med to coarse grain, fine to coarse Gravel.				
		126	Cobbles.				
		57	Dense, Lt Brown, Gravelly Sand (SP) fine to coarse grain, fine to coarse gravel				
		40					
	▼	654	Boulder.				
		730					
15		147	Cobbles or Boulders, small amount Sand, (SP) medium to coarse grain.				
		153					
		341	Sandy Dense, Lt Brown, Sandy Gravel (GP) med to coarse grain, fine to				
		116	coarse gravel. interniced with cobbles or boulders, angular				
		65	on one side, granite in nature, transported.				
20		162	Lt, Red-Brown				
		226					
		110					
		128					
		117					
25		568	Cobbles or Boulder				
		106	Lt Brown				
		164					
		120					
		127	Gray-Brown, Light.				
30		100					
		100	med				
		57	Dense, Lt Brown, Gravelly Sand (SP) med to coarse grain, fine to coarse				
		80	gravel, occasional cobbles, angular on one side, granite				
		70					
35		63					
		99.					
		81	Lt. Gray-Brown				
		73					
		95	Lt Brown				
40		52					
		61					
		78					
		73					
		84					
45		40					
		21					
		42					
		37					
50		95					

Project: Mentone Dam
Project No. 4174SC

LOG OF BORING 6 (six)

Fig.

DATE OF BORING 16 October 1984 WATER DEPTH 13.5ft DATE MEASURED 10-16-84
TYPE OF DRILL RIG Becker Reverse Circulation HOLE DIAMETER 7 ins
WEIGHT OF HAMMER 8000 ft lb/in² FALLING _____ SAMPLES _____

DEPTH, FT.	SAMPLES	BLOWS/FOOT	DESCRIPTION	UNC. COMP. STRENGTH, tsf	MOISTURE CONTENT, %	DRY DENSITY pcf	OTHER TESTS
			SURFACE ELEVATION:				
	73		Dense, Lt Brown Gravelly Sand (SP) med to coarse grain, fine to coarse Gravel, occasional cobbles, angular on one side, granitic in nature.				
	105						
	93		Dense Lt Brown Sand (SP) med to coarse grain, fine gravel.				
	127						
5	290		Boulder 54ft to 57ft.				
	372						
	461						
	117		Dense Lt Brown Sand (SW) fine to coarse, fine to coarse gravel occasional cobbles, angular on one side, granitic in nature				
	81						
10	115						
	113						
	70						
	48						
	127						
15	132						
	110						
	100						
	104						
	78						
20	75		Cobble or Boulder				
	204						
	60						
	121						
	108						
25	85		Dense, Lt, Brown Sand (SW) fine to coarse, fine gravel occasional coarse gravel.				
	85						
	83						
	140						
	336						
30	6"		78.5ft Dark Grey weathered granitic cuttings, high biotite content easily crumbled, possible contact with bedrock		1500 Hrs.		
			Bottom of Hole at 79ft.				
35							
40							
45							
50							

Project: Mentone Dam
Project No. 41745C

LOG OF BORING 6

Fig.

DATE OF BORING 17 October 1964 WATER DEPTH _____ DATE MEASURED 10-17-64
TYPE OF DRILL RIG Becker Reverse Circulation HOLE DIAMETER 7 inch
WEIGHT OF HAMMER 8000 ft lbs/in² FALLING _____ SAMPLES _____

2067.4?
17.5
2-50

DEPTH, FT.	SAMPLES	BLOWS/FOOT	DESCRIPTION	UNC. COMP. STRENGTH, ksf	MOISTURE CONTENT, %	DRY DENSITY pcf	OTHER TESTS
			Start Drilling 0917 hrs.				
			SURFACE ELEVATION:				
		26	Med Dense, Moist, Brown, Silty Sand, (SM) fine to medium grain.				
		102	Dense, Damp, Lt Brown, Sand (SP) medium to coarse grain, fine to coarse gravel with cobbles or boulders				
		99					
		280	Cobbles or Boulders from 2ft. to 14ft.				
5		100					
		144					
		44	Dense Damp Lt Brown Sand (SP) ~ 1ft				
		114					
		111					
		91	Dense Damp, Lt Brown Sand (SP) ~ 3ins				
10		70					
		139					
		141					
		114					
		180	Cobbles or Boulders 14ft to 16.5ft				
15		192					
		122	Dense, Lt Brown Gravelly Sand (SP) medium to coarse grain				
	▼	96	fine to coarse Gravel, with Cobbles or Boulders, angular				
		40	on one side, granitic in nature, transported.				
20		71					
		104					
		72					
		95					
		68	Lt Red-Brown				
25		125					
		77					
		523	Boulder, granitic in nature, transported.				
		1960	26ft to 29.5ft.				
		1643					
		233	Dense, Lt Brown, Gravelly Sand (SP) coarse grain, fine to coarse				
30		132	Gravel, occasional Cobbles, angular on one side				
		64	Granitic in nature, transported.				
		68					
		48					
		33	Sands fine to coarse grain				
35		31					
		67					
		47					
		43					
		52					
40		55					
		63					
		77					
		96					
		124					
45		74					
		46					
		300					
		327					
50		91					

Project: Mentone Dam
Project No. 41745C

LOG OF BORING 7

Fig.

DATE OF BORING 17 October 1984 WATER DEPTH _____ DATE MEASURED 10-17-84

TYPE OF DRILL RIG Becker Reverse Circulation HOLE DIAMETER 7 inch

WEIGHT OF HAMMER 8000 ft lbs/in² FALLING _____ SAMPLES _____

DEPTH, FT.	SAMPLES	BLOWS/FOOT	DESCRIPTION	UNC. COMP. STRENGTH, ksf	MOISTURE CONTENT, %	DRY DENSITY pcf	OTHER TESTS
50			SURFACE ELEVATION:				
		58	Dense, Lt. Brown Gravelly Sand (SP), fine to coarse grain, fine to				
		74	coarse Gravel, occasional Cobbles, granitic in				
		87	nature, angular on one side, transported.				
		61					
55	18	79					
		61					
		49					
		57					
		68					
60	18	73					
		210					
		94	More Cobbles,				
		72					
		128					
65	18	82					
		400					
		484	Boulder, granitic in nature, angular on one side, transported				
		713	from 65.5 ft to 68.5 ft.				
		155					
70	30	124	Dense, Lt Brown Sandy Gravel (GP) coarse grain Sand, fine to coarse				
		94	Gravel,				
		197					
		2395	Boulder, granitic in nature, angular on one side, transported				
		264	from 71.5 ft to 74 ft.				
75	28	125	Dense, Lt Brown, Sandy Gravel (GP) med to coarse grain sand, fine to				
		140	coarse Gravel with Cobbles or Boulders, angular on one side				
		93	granitic in nature, transported.				
		93	Lt Gray 6"				
		105	Lt Brown				
80	30	155					
		62					
		150					
		277					
		114					
85	35	370					
			85 ft Dark Gray weathered granitic cuttings, high biotite content				
			easily crumbled, possible contact with bedrock.		complete drilling		
			Bottom of Hole 85 ft.		at 2.45 hrs.		
90							
95							
100							

Project: Mentone Dam
Project No. 41745C

LOG OF BORING 7

Fig.

DATE OF BORING 18 October 1984 WATER DEPTH 20.5 ft DATE MEASURED 10-18-84
TYPE OF DRILL RIG Becker Reverse Circulation HOLE DIAMETER 7 ins (Pilot Hole)
WEIGHT OF HAMMER _____ FALLING _____ SAMPLES _____

DEPTH, FT.	SAMPLES	BLOWS/FOOT	DESCRIPTION	UNC. COMP. STRENGTH, ksf	MOISTURE CONTENT, %	DRY DENSITY pcf	OTHER TESTS
			Start Drilling at 1010 Hrs.				
			SURFACE ELEVATION: 0				
		Push	Loose, Damp, Brown, Sand (SP) fine to medium grain, some silt.				
		605	Boulder, Granitic in nature, angular on one side, transported				
		581					
		200	Cobbles or Boulders, angular on one side, transported, much rock flour, no sand				
5		195					
		90					
		69					
		97					
		102					
		81					
10		142	← 11ft about 2 inch loose Sand (SP) fine grain				
		108					
		86					
		410					
		128					
15		52					
		47	Dense, Moist, Brown, Sand (SW) fine to coarse grain, fine to coarse Gravel				
		27					
		40					
		47					
20	▼	75					
		67					
		71	Lt. Brown				
		81					
		94					
25		173	Red-Brown, Cobble or Boulder				
		74					
		65					
		106	Lt. Brown				
		50					
30		56					
		32	Dense, Lt Brown, Gravelly Sand (SP) med to coarse grain, fine to coarse Gravel, occasional Cobbles or Boulders, angular on one side, granitic in nature, transported				
		49					
		30					
35		26					
		75					
		106					
		93					
		51					
40		44					
		55	Dense, Lt. Brown, Sandy Gravel (GP) med to coarse Sand, fine to Coarse Gravel, occasional Cobbles or Boulders, angular on one side, granitic in nature, transported.				
		80					
		40					
		55					
		300					
45		162	Boulder, Granitic, angular on one side, transported.				
		6"	Driller abandoned hole at 45.5 ft. Time 1640 Hrs				
50							

At 13ft Drill Stem almost 10° from vertical. Driller made four attempts, then Drill Stem near vertical, continued drilling.

Project: Mentone Dam	LOG OF BORING 8	Fig.
Project No. 41745C		

DATE OF BORING 19 October 1984 WATER DEPTH _____ DATE MEASURED _____

TYPE OF DRILL RIG Becker Reverse Circulation HOLE DIAMETER 7 ins

WEIGHT OF HAMMER 8000 ft lbs/in² FALLING _____ SAMPLES _____

DEPTH, FT.	SAMPLES	BLOWS/FOOT	DESCRIPTION (logged by SD)	UNC. COMP. STRENGTH, ksf	MOISTURE CONTENT, %	DRY DENSITY pcf	OTHER TESTS
			SURFACE ELEVATION:				
		90	Loose, damp, brown, very fine-med. grained sand				08
5		200	Boulder; bits of rock flour refusal at 5'				
			TD 5'2"				
10							
15							
20							
25							
30							
35							
40							
45							
50							

Project: Mentone Dam.
Project No. 4174 SC

LOG OF BORING 8A

Fig.

LA/OR-0181-204

WOODWARD - CLYDE CONSULTANTS

DATE OF BORING 10/1984 WATER DEPTH _____ DATE MEASURED _____

TYPE OF DRILL RIG Bucket Rotary Core HOLE DIAMETER 7in

WEIGHT OF HAMMER _____ FALLING _____ SAMPLES _____

DEPTH, FT.	SAMPLES	BLOWS/FOOT	DESCRIPTION	UNC. COMP. STRENGTH, ksf	MOISTURE CONTENT, %	DRY DENSITY pcf	OTHER TESTS
			logged by SL				
			SURFACE ELEVATION:				Tin:
			loose, damp, brown fine-coarse grained sand				1135
		100					
		700	Boulder lots of rock Dam				
		125	Cobbles				
5		50	light tan, loose, fine to med grained sand and gravel				
		60					
		50					
		200	Cobbles				
10		100	pebbles? hole not vertical				
			TD 11'				
15							
20							
25							
30							
35							
40							
45							
50							

Project: Mentua Dam
Project No. 91795C

LOG OF BORING 8B

Fig.

DATE OF BORING 2 November 1984 WATER DEPTH 20 ft DATE MEASURED Drillers note

TYPE OF DRILL RIG Becker Reverse Circulation HOLE DIAMETER 7 inches

WEIGHT OF HAMMER 8000 ft lbs/in² FALLING _____ SAMPLES C 6 (abandoned)

DEPTH, FT.	SAMPLES	BLOWS/FOOT	DESCRIPTION	UNC. COMP. STRENGTH, ksf	MOISTURE CONTENT, %	DRY DENSITY pcf	OTHER TESTS
			SURFACE ELEVATION:				
20	▼		Saturated. (Drillers note)				
45	36 60 55 97 105		Saturated, Dense, Lt Brown, Gravelly Sand (SP) fine to medium grain, fine to coarse gravel occasional Cobbles or Boulders, granitic in nature, transported.				

○ 7 (Long Dia).
○ 6
● 9
N

Project: Mantone Dam.
Project No. 41745C

LOG OF BORING 9

Fig.

DATE OF BORING 3 November 1984 WATER DEPTH 20ft DATE MEASURED drillers note

TYPE OF DRILL RIG Becker Reverse Circulation HOLE DIAMETER 7 inch

WEIGHT OF HAMMER 8000 ft lbs/in² FALLING _____ SAMPLES _____

DEPTH, FT.	SAMPLES	BLOWS/FOOT	DESCRIPTION	UNC. COMP. STRENGTH, ksf	MOISTURE CONTENT, %	DRY DENSITY pcf	OTHER TESTS
			SURFACE ELEVATION:				
50		143	Dense, Saturated, Lt Brown Sandy Gravel (GP) med to coarse grain sand, fine to coarse gravel, Cobbles or Boulders, granitic in nature, transported				
		91					
		131					
		200					
55	8'	133					
		127					
		113					
		78	Dense, Lt Brown Gravelly Sand (SP) fine to med grain, fine to coarse gravel				
		136	Dense, Lt Brown Sandy Gravel (GP), med to coarse grain sand fine to coarse gravel, Cobbles or Boulders, granitic in nature, transported.				
60	18'	176					
		110					
		169					
		197	Yellow Brown				
		155	Lt. Brown				
65	13'	135					
		160					
		122					
		71					
		120					
70	28'	222	Cobbles or Boulders, granitic in nature, transported.				
		175					
		106					
		90					
		90					
75	25'	124	Dense, Lt Brown, Gravelly Sand (SP) med to coarse grain, fine to coarse				
		125	Gravel, Cobbles or Boulders, granitic in nature, transported.				
		88	Lt. Gray				
		164					
		72					
30		186	Contact with possible bedrock Granitic, high biotite content, easily crumbled, slightly sticky gray in color.				
			Bottom of Boring at 80 ft.				
35							
40							
45							
50							

Project: Mentone Dam
Project No. 41745C (DRIL)

LOG OF BORING 9

Fig.

DATE OF BORING 03 November 1984 WATER DEPTH 9 feet DATE MEASURED 03 Nov. 1984
TYPE OF DRILL RIG Becker Reverse Circulation HOLE DIAMETER 7 inches
WEIGHT OF HAMMER 8000 ft lbs/in² FALLING _____ SAMPLES _____

DEPTH, FT.	SAMPLES	BLOWS/FOOT	DESCRIPTION WCC Boring 10, Becker Boring 12	UNC. COMP. STRENGTH, ksf	MOISTURE CONTENT, %	DRY DENSITY pcf	OTHER TESTS
			SURFACE ELEVATION:				
		22 96 151 279	Dense, Damp, Lt. Brown, Sandy Gravel (GP), fine to coarse grain, fine to coarse gravel.				
		115	Cobbles or Boulder, granitic in nature, fractured on one side transported.				
5		44 41 77 70	Dense, Damp, Lt. Brown, Sand (SP), fine to medium grain occasional fine to coarse gravel.				
10		160 186	Cobbles or Boulder, granitic in nature, fractured on one side transported.				
		113 59 67 67	Dense, Lt. Brown, Sandy Gravel (GP), coarse sand, fine to coarse gravel, cobbles.				
15		109 103 110 100	Dense, Damp, Lt. Brown, Sand (SP), fine to medium gram. Occasional fine to coarse gravel.				
20		61 180 197	Dense, Damp, Lt. Yellow Brown, Sandstone	Similar to Bedrock Sandstone outcrop at East side of canyon.			
25							
30			05 Nov Tricone to 28 ft. Driller Abandon.				
35							
40							
45							
50							

Project: Mentone Dam
Project No. 41745C

LOG OF BORING 10

Fig.

DATE OF BORING 5 November 1964 WATER DEPTH _____ DATE MEASURED _____

TYPE OF DRILL RIG Becker Reverse Circulation HOLE DIAMETER 7 ins

WEIGHT OF HAMMER 8000 ft lbs/in^2 FALLING _____ SAMPLES _____

DEPTH, FT.	SAMPLES	BLOWS/FOOT	DESCRIPTION	UNC. COMP. STRENGTH, ksf	MOISTURE CONTENT, %	DRY DENSITY pcf	OTHER TESTS
			WCC Boring 11 Becker Boring 13				
2072.9			SURFACE ELEVATION:				
5			Not logged				
10							
15							
			Bottom of Hole @ 16.5 ft.				
20							
25							
30							
35							
40							
45							
50							

Project: Mentone Dam
Project No. 41745C

LOG OF BORING 11

Fig.

DATE OF BORING 6 November 1984 WATER DEPTH _____ DATE MEASURED _____

TYPE OF DRILL RIG Becker Reverse Circulation HOLE DIAMETER 7 ins.

WEIGHT OF HAMMER 8000 ft lbs/in² FALLING _____ SAMPLES _____

DEPTH, FT.	SAMPLES	BLOWS/FOOT	DESCRIPTION	UNC. COMP. STRENGTH, ksf	MOISTURE CONTENT, %	DRY DENSITY pcf	OTHER TESTS
			WCC Boring 12 Becker Boring 14				
			SURFACE ELEVATION:				
5			Not logged				
10							
15			Bottom of Hole @ 13 ft				
20							
25							
30							
35							
40							
45							
50							

Project: Mentone Dam
Project No. 41745C

LOG OF BORING 12

Fig.

LA/OR-0181-204

WOODWARD - CLYDE CONSULTANTS

DATE OF BORING 6 November 1984 WATER DEPTH _____ DATE MEASURED _____

TYPE OF DRILL RIG Becker Reverse Circulation HOLE DIAMETER 7ins

WEIGHT OF HAMMER 8000 ft lbs/in² FALLING _____ SAMPLES _____

DEPTH, FT.	SAMPLES	BLOWS/FOOT	DESCRIPTION	UNC. COMP. STRENGTH, ksf	MOISTURE CONTENT, %	DRY DENSITY pcf	OTHER TESTS
			WCC Boring 13 Becker Boring 15				
			SURFACE ELEVATION:				
5							
10							
15							
20			Bottom of Hole @ 19 ft.				
25							
30							
35							
40							
45							
50							

Project: Mentone Dam
Project No. 41745C

LOG OF BORING 13

Fig.

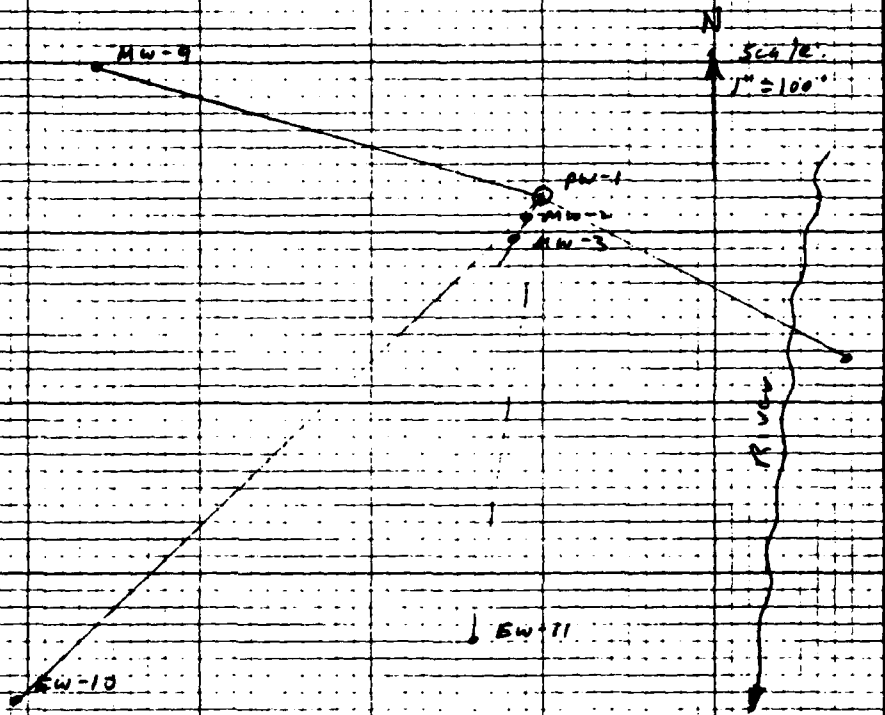
PREPARED BY:	DATE:	Woodward-Clyde Consultants	PROJECT NO.:	TASK:
TS	11/15/84	ORANGE COUNTY, CALIFORNIA	41745C	PUMP
CHECKED BY:	DATE:		PROJECT NAME:	
		Mentone Dam	Mentone Dam	
		Aquifer Test Analysis		SHEET: 1 OF: 4

REFERENCE:

Aquifer Test Consisted of Rest period, Pumping, Recovery : 11/7 - 10/84.

Test data Summarized on Table 5

Well locations shown below

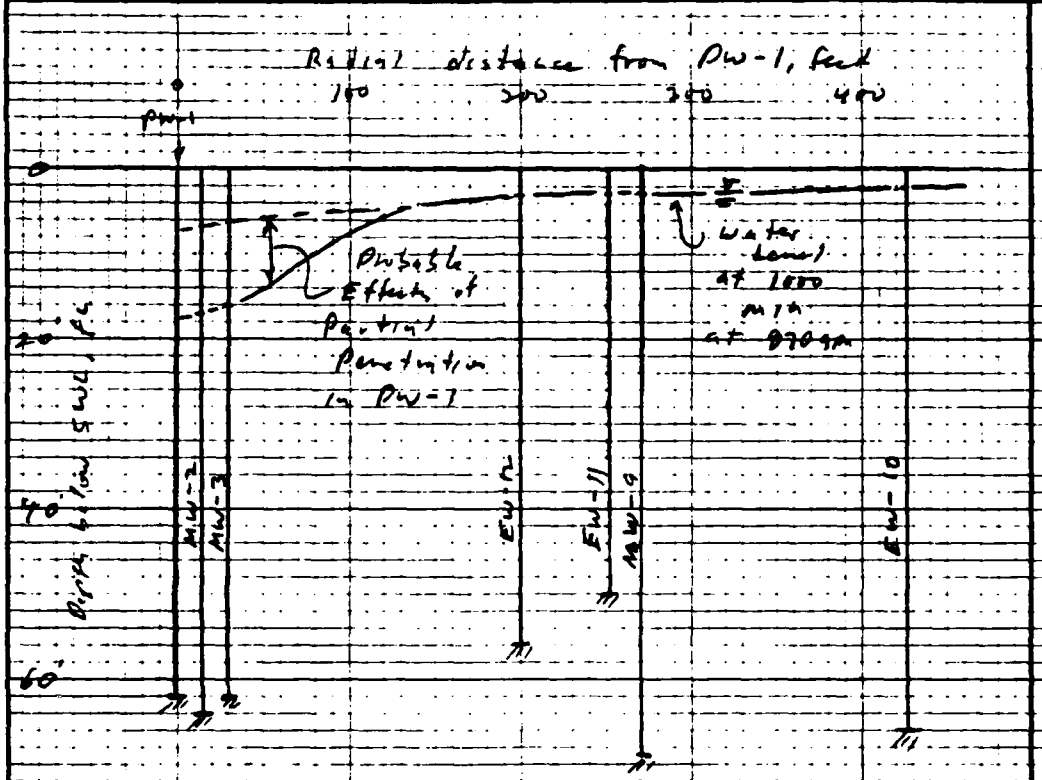


Pertinent well data

Well No.	AH A	ft from PW-1	Total Depth	Depth to Sand	Saturated Thickness of Aquifer
PW-1	B-11	0.5 ft	74 ft	12 ft	62 ft
MW-2	B-6	35	78	14	64
MW-3	B-9	30	74	12	62
MW-9	B-2	271	93	24	69
EW-12	AB-4	428	78	12	66
EW-11	AB-6	255	61	11	50
EW-10	AB-11	200	64	8	56

PREPARED BY:	DATE:	Woodward-Clyde Consultants ORANGE COUNTY, CALIFORNIA	PROJECT NO.:	TASK:
RB	11/15		41745C	Rep
CHECKED BY:	DATE:		PROJECT NAME:	
		Ad Test		SHEET: 3 OF: 4

REFERENCE:



C-1 Appears additional 10' of L E MW-2 + MW-3 Drops due to Partial Penetration into PW-1.

C-2 Draw for other OW's gives $\Delta s = 3-3.5'/\log$

C-3 Use these data to Calc. T, Q's

4. Calc. Transmissivity

1. From s-t data

$Q = 870$ gpm; $\Delta s/\log t = 1.73'/\log t$

$T = \frac{264\,Q}{\Delta s/\log t} = \frac{264 \times 870}{1.73} = 133{,}000$ gpd/ft

GW Tech. Fundamentals 1966 Eq 7

2. From s-r data

$Q = 870$ gpm; $\Delta s/\log r = 3.5'/\log r$

$T = \frac{528\,Q}{\Delta s/\log r} = \frac{528 \times 870}{3.5} = 131{,}000$ gpd/ft

Eq 8

Good Agreement.

LA/OR-0781-400

TIMES AND DEPTHS TO WATER (DTW)[A]

Date	PW-1 Time	PW-1 psi[F]	MW-2 Time	MW-2 DTW	MW-3 Time	MW-3 DTW	MW-9 Time	MW-9 DTW	EW-10 Time	EW-10 DTW	EW-11 Time	EW-11 DTW	EW-12 Time	EW-12 DTW	Flow Meter Readings (Acre-Feet x.001) Time	Flow Meter Readings (Acre-Feet x.001) Meter	Comments
November	0945	18.5	0951	13.82	0950	11.78	0943	23.82	0939	11.76	0936	10.51	0910	7'-6'			-Static, non-pumping
8,9,&10	1015	10.5	1015	-	1015	-	1015	-		-	1015	-	1015	-	1015	09739	conditions
	1016	9.5	1015.15	15.49	1016	20.15	1028.40	24.09	1019.2	11.80	1017	10.51	1034	8'-0	1017	09745	-Start pumping at 1015
	1017	9.25	1017	21.76	1017.45	21.68	1050.10	24.25	1024.2	11.86	1021.4	10.55	1043	8'-2	1018	09748	on 11/8/84
	1018	9.00	1018.45	24.36	1019.25	22.17	1148	24.75	1037.1	12.02	1031.5	10.62	1139.1	8'-6.5	1019	09750	-Testing ends at 0800 on
	1020	8.75	1020.15	25.34	1021.30	22.44	1231	24.55	1042.8	12.04	1039.8	10.70	1245	8'-8.7	1020	09753	11/10/84
	1021	8.5	1022.20	25.70	1023.10	22.61	1341	24.75	1047.9	12.08	1045.2	10.72	1353	8'-11	1040	09850	
	1022	8.5	1023.45	25.56	1025.20	22.70	1437	24.80	1053.3	12.10	1050.9	10.76	1448	8'-10.5	1054	09887	
	1023	8.5	1024.30	25.66	1031.45	22.97	1549	25.05	1058.4	12.12	1055.8	10.79	1604	9'-4	1104	10039	
	1024	8.5	1033.40	26.17	1034.15	23.01	1648	25.30	1104.0	12.15	1101.5	10.83	1658	9'-5	1329	10280	
	1025	8.5	1035.10	26.14	1036.10	23.03	1752	25.45	1109.8	12.20	1107.2	10.88	1806	9'-7	1426	10377	
	1030	8.5	1040.30	26.25	1039.45	23.19	1839	25.60	1236	12.46	1240	11.34	1852	9'-9	1538	10579	
	1040	8.5	1045.00	26.29	1046	23.33	2021	25.87	1345	12.57	1313	11.62	2035	10'-0	1636	10740	
	1045	8.5	1053.45	26.40	1055	23.42	2153	26.10	1441	12.56	1347	11.60	2158	10'-3	1734	10901	
	1100	8.25	1107.30	26.56	1106	23.65	2353[C]	26.45	1556	12.76	1444	11.67	2400[C]	10'-6	1826	11044	
	1115	8.00	1113	26.68	1117.30	23.80	0155	26.77	1652	12.87	1600	11.92	0210	10'-9	2000	11308	
	1130	8.25	1143.30	26.91	1139	23.85	0355	27.09	1758	13.00	1655	12.08	0401	10'-11.5	2145	11596	
	1200	8.00	1227	27.30	1222	24.10	0552	27.22	1844	13.80	1801	12.30	0557	11'-1	2345[C]	11923	
	1230	8.25	1333	27.60	1336	24.6	0757	27.50	2025	13.29	1847	12.42	0811	11'-3	0145	12243	
	1300	8.00	1424	26.55	1428	24.10	0853	27.70	2205	13.45	2031	12.69	0905	11'-4.5	0345	12556	
	1330	7.75	1542	28.05	1545	25.15	1025	27.90	2354[C]	13.67	2201	12.97	1038	11'-6.5	0545	12867	
	1402	B	1640	28.95	1642	25.50	1150	28.15	0200	13.86	2357[C]	13.22	1204	11'-8.5	0745	13146	
	1430	7.75	1740	28.70	1743	25.80	1322	28.35	0410	14.12	0204	13.51	1332	11'-10	0843	13309	
	1500	7.75	1830	29.15	1833	26.10	1445	28.50	0600	1432	0405	13.75	1456	11'-11.5	1013	13550	
	1530	7.5	2018	29.64	2012	26.50	1815	28.90	0804	14.49	0603	14.00	1815	12'-3.2	1143	13796	
	1600	7.5	2146	30.00	2148	26.99	1915	29.05	0858	14.60	0807	14.16	1903	12'-4.2	1313	14039	
	1630	7.25	2344[C]	30.39	2347[C]	27.40	2000[E]	NR	1029	14.78	0900	14.26	2000[E]	NR	1438	14268	
	1700	7.25	0147	30.62	0150	24.64	2105	28.34	1156	14.95	1032	14.45	2006.5	12'-4	1806	14833	
	1730	7.00	0346	30.91	0349	27.85	2140	28.02	1326	15.12	1200	14.63	2016.1	12'-2	1904	14990	
	1800	6.75	0546	29.55	0549	26.64	2205	27.85	1449	15.26	1328	14.81	2024.3	12'-0.5	2000[E]	-	
	1830	6.50	0749	31.03	0751	27.90	2235	27.70	1805	15.63	1452	14.97	2034.3	11'-11			
	1900	6.25	0846	31.50	0848	28.40	2315	27.48	1908	15.74	1808	15.33	2041.0	11'-10			
	1930	6.25	1018	31.75	1020	28.75	2355[D]	27.31	2000[E]	-	1905	15.45	2049.1	11'-9			
	2000	6.00	1144	32.00	1146	28.90	0115	26.94	2001	15.80	2000[E]	NR	2056.1	11'-8			
	2030	6.00	1315	32.30	1317	29.15	0215	26.85	2011	15.81	2003.9	15.52	2104.5	11'-7			
	2100	6.00	1440	32.45	1442	29.40	0315	26.74	2019.6	15.80	2013.5	15.52	2155	11'-2			
	2130	6.00	1809	32.95	1811	29.90	0415	26.59	2027.5	15.79	2021.7	15.49	2300[D]	10'-9.5			
	2200	6.00	1908	33.10	1911	30.00	0515	26.38	2036.9	15.77	2031.4	15.49	0009	10'-6			
	2230	6.00	2000[E]	NR	2000[E]	30.06	0615	26.31	2048.8	15.77	2038.8	15.45	0103	10'-3.5			
	2300	6.0	2016.5	23.29	2000.10	27.75	0715	26.20	2051.8	15.77	2046.5	15.40	0155	10'-1			
	2330[C]	5.5	2019	23.09	2000.50	25.36	0815	26.12	2059.5	15.73	2054.2	15.36	0255	9'-11			
	0000	5.5	2022	22.91	2001.15	24.69			2150	15.55	2102.1	15.32	0355	9'-9			
	0030	5.5	2024	22.80	2002.30	23.85			2307[D]	15.32	2153	15.00	0455	9'-7			
	0100	5.5	2027	22.60	2003	23.51			0001	15.22	2305[D]	14.63	0555	9'-5			
	0130	5.5	2029	22.42	2003.40	23.34			0055	15.14	0004	14.37	0655	9'-4			
	0200	4.75	2031	22.34	2004.10	23.23			0202	15.06	0059	14.15	0755	9'-3			
	0230	5.0	2036	22.08	2004.45	23.08			0302	14.95	0159	14.00					
	0300	5.0	2041	21.82	2005.25	22.95			0402	14.89	0259	13.80					
	0330	5.0	2046	21.60	2006	22.73			0502	14.82	0359	13.69					
	0400	5.0	2051	21.48	2007	22.49			0602	14.75	0459	13.54					

TIMES AND DEPTHS TO WATER (DTW)[A]

Date	PW-1 Time	PW-1 psi[F]	MW-2 Time	MW-2 DTW	MW-3 Time	MW-3 DTW	MW-9 Time	MW-9 DTW	EW-10 Time	EW-10 DTW	EW-11 Time	EW-11 DTW	EW-12 Time	EW-12 DTW	Flow Meter Readings (Acre-Feet x.001) Time	Flow Meter Readings (Acre-Feet x.001) Meter	Comments
	0430	5.0	2056	21.25	2008	22.29			0702	14.64	0559	13.44					
	0500	5.0	2101	21.00	2009	22.17			0802	14.62	0659	13.35					
	0530	5.5	2111	20.62	2010	21.85					0759	13.23					
	0600	5.5	2126	20.35	2011	21.83											
	0630	5.5	2132	20.00	2012	21.71											
	0700	5.5	2145	19.78	2013	21.59											
	0730	G	2151	19.63	2014	21.46											
	0800	6.25	2201	19.48	2015	21.42											
	0830	5.75	2231	19.08	2018	21.17											
	0900	5.50	2311	18.66	2021	21.04											
	0930	5.25	2351[D]	18.36	2023	20.82											
	1000	5.00	0111	17.92	2025	20.69											
	1030	5.00	0206	17.77	2028	20.53											
	1100	5.25	0306	17.52	2030	20.36											
	1130	5.50	0406	17.31	2035	20.14											
	1200	5.00	0506	17.09	2040	19.90											
	1230	5.00	0606	16.96	2045	19.67											
	1300	5.00	0706	16.85	2050	19.48											
	1330	5.00	0806	16.69	2055	19.31											
	1400	5.00			2100	19.10											
	1430	4.75			2110	18.63											
	1500	4.75			2120	18.30											
	1530	4.75			2130	18.04											
	1600	4.75			2140	17.65											
	1630	4.75			2150	17.59											
	1700	4.75			2200	17.45											
	1730	4.50			2230	17.05											
	1800	4.50			2310	16.58											
	1830	4.50			2350[D]	16.34											
	1900	4.50			0110	15.82											
	1930	4.50			0205	15.66											
	2000[E]	4.50			0305	15.46											
	2000.30	11.50			0405	15.27											
	2001	12.00			0505	15.09											
	2001.30	13.00			0605	14.97											
	2002	13.00			0705	14.82											
	2003	13.00			0805	14.67											
	2004	13.25															
	2005	13.50															
	2006	13.50															
	2007	13.75															

TIMES AND DEPTHS TO WATER (DTW)[A]

Date	PW-1 Time	PW-1 psi[F]	MW-2 Time	MW-2 DTW	MW-3 Time	MW-3 DTW	MW-9 Time	MW-9 DTW	EW-10 Time	EW-10 DTW	EW-11 Time	EW-11 DTW	EW-12 Time	EW-12 DTW	Flow Meter Readings (Acre-Feet x.001) Time	Flow Meter Readings (Acre-Feet x.001) Meter	Comments
	2008	13.75															
	2009	13.75															
	2010	14.00															
	2015	14.00															
	2020	14.25															

NOTES:
A - All depths are measured in feet below top of casing.
B - Pump shut down due to clutch slippage. Restart at same pumping rate.
C - End 11/8/84.
D - End 11/9/84.
E - End Pump test (begin recovery).
F - Air line gauge readings.
G - Work on motor - only running on 5 cylinders for the past 4 hours.

GEOTECHNICAL

ATTACHMENT A-4

Leighton and Association Geophysical Report

LEIGHTON and ASSOCIATES

INCORPORATED

SOIL ENGINEERING GEOLOGY GEOPHYSICS GROUND WATER HAZARDOUS WASTES

SEISMIC REFRACTION INVESTIGATION AT THE PROPOSED UPPER SANTA ANA DAM SITE, SANTA ANA RIVER, NEAR REDLANDS, CALIFORNIA

May 11, 1984

Project No. 1840081-01

Prepared for:

U. S. ARMY CORPS OF ENGINEERS
Foundations and Materials Branch
300 N. Los Angeles Street
P.O. Box 2711
Los Angeles, California 90053
Attention: Mr. David W. Lukesh

1151 DURYEA AVENUE, IRVINE, CALIFORNIA 92714 (800) 253-4567 • (714) 250-1421 • (213) 691-2125

IRVINE • WESTLAKE/VENTURA • DIAMOND BAR/WALNUT • SAN BERNARDINO/RIVERSIDE • SAN DIEGO
PALM DESERT • SANTA CLARITA/VALENCIA

LEIGHTON and ASSOCIATES
INCORPORATED

SOIL ENGINEERING GEOLOGY GEOPHYSICS GROUND WATER HAZARDOUS WASTES

May 11, 1984

Project No. 1840081-01

TO: U. S. Army Corps of Engineers
Foundation and Materials Branch
300 N. Los Angeles Street
P.O. Box 2711
Los Angeles, California 90053

ATTENTION: Mr. David W. Lukesh

SUBJECT: Seismic Refraction Investigation at the Proposed Upper Santa Ana Dam Site, Santa Ana River, Near Redlands, California

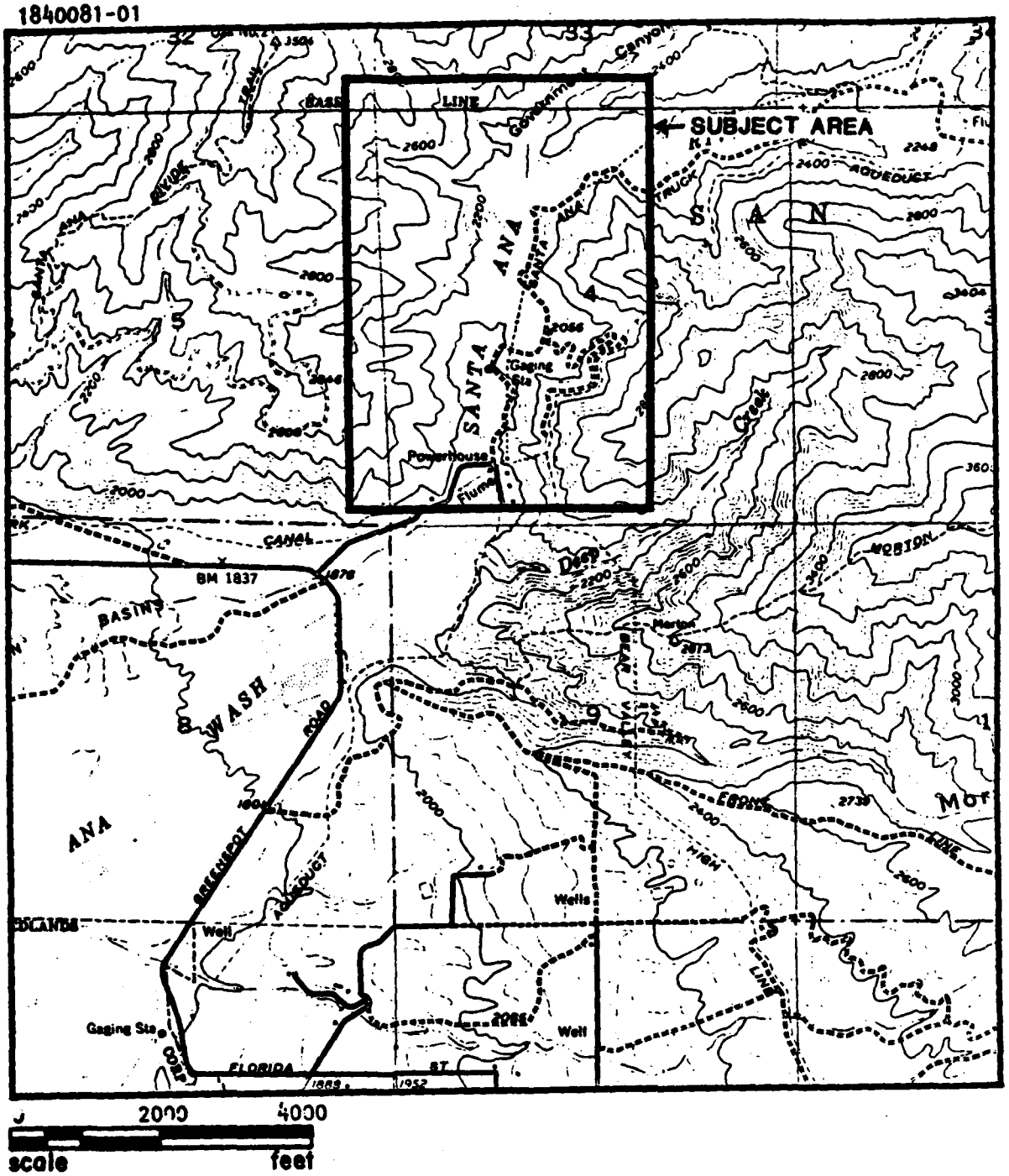
Leighton and Associates, Inc. is pleased to submit our report describing the results of our seismic refraction survey at the proposed alternate Upper Santa Ana Dam site, Santa Ana River, near Redlands, California. Four seismic refraction lines were surveyed for the purpose of determining the depth to bedrock beneath the alluvium in the river channel.

The bedrock surface beneath the river channel is interpreted from three seismic lines to have a generally uniform slope toward the west side of the river channel where the bedrock appears to make a fairly sharp join with the steep canyon wall. Depth to bedrock beneath the alluvium ranges from 50 to 60 feet near the east side of the channel to 75 to 100 feet near the west side. The fourth seismic line, oriented essentially parallel to the river channel, indicates a gently sloping bedrock surface toward the mouth of the river; depth to bedrock below this line ranges from 70 to 80 feet.

Accompanying Maps and Illustrations

Figure 1 - Index Map - Page 2
Figure 2 - Locations of Seismic Lines - Rear of Text
Figure 3 - Seismic Line 1 - Rear of Text
Figure 4 - Seismic Line 2 - Rear of Text
Figure 5 - Seismic Line 3 - Rear of Text
Figure 6 - Seismic Line 4 - Rear of Text
Table 1 - Summary of Geophysical Interpretations - Page 4

1840081-01



INDEX MAP,

PROPOSED SITE OF UPPER SANTA ANA DAM,

SANTA ANA RIVER

Base Map: Yucaipa, California

LEIGHTON and ASSOCIATES
INCORPORATED

Introduction

A seismic refraction investigation was conducted at a proposed site for the Upper Santa Ana Dam near Redlands, California (Figure 1), on April 12 and May 3, 1984. The purpose of this investigation was to determine the depth to bedrock beneath the alluvium near the mouth of the Santa Ana River, near Redlands, California. A letter report was submitted previously that included our preliminary interpretations of the data collected on April 12, 1984 (Leighton and Associates, April 16, 1984). The present report includes our interpretations of the May 3, 1984 investigation and incorporates interpretations from our previous survey.

Field Operations

The seismic refraction survey consisted of three lines oriented across the river channel (Figure 2); Lines 1 and 3 were 550 feet long and Line 2 was 500 feet long. Line 4, 550 feet long, was oriented within and essentially parallel to the river channel (Figure 2). A GeoMetrics Nimbus Model 1210F signal enhancement seismic refraction/reflection seismograph was used with twelve geophones spaced 50 feet apart. At some geophone stations, two geophones were connected in parallel to increase the signal to noise ratio and to improve the quality of the field data. Energy to produce compressional seismic waves was generated by a 16-pound sledge hammer striking an aluminum plate at each end and at, or near, the center of all survey lines. An explosive source was used for in-line offset shots for the survey oriented parallel to the river. Field data were viewed in the field on a CRT display to determine the quality of the data. A permanent record of the data was produced on electrosensitive paper using the instrument's oscillograph.

Interpretations

Arrival times of the compressional seismic waves at the geophones from the energy source were determined from the paper record, corrected for elevation, and analyzed in the office. The time-distance graphs for the four survey lines are shown on Figures 3 through 6; the geophysical analysis and inferred geological interpretations of the geophysical data are also shown on these figures.

The data from the seismic surveys indicate four layers; their velocities and inferred geologic interpretations are summarized in Table 1.

TABLE I

SUMMARY OF GEOPHYSICAL INTERPRETATIONS

Layer	Velocity Range (ft/sec)	Depth to Base (ft)	Thickness (ft)	Inferred Geologic Interpretation
1	(a) 1,600-1,800 (b) 3,200-4,000	0-15 0-30	0-15 0-30	Unsaturated alluvium; higher velocities may be caused by concentrations of boulders.
2	2,500	0-10	0-10	Talus, colluvium, or debris from steep canyon walls on west side of canyon.
3	6,000-8,500	50-100	50-90	Saturated alluvium; higher velocities may be caused by concentrations of boulders.
4	10,000-15,000 average, 12,000	——	——	Bedrock; granite, gneiss; range in velocity results primarily from sloping bedrock surface which results in apparent velocities.

The range of velocities in Layers 1 and 3, unsaturated and saturated alluvium, respectively, is likely to be caused by the heterogeneous character of these materials. The alluvium contains clastic materials ranging in size from sand to boulders. Areas where concentrations of sands and pebbles occur would produce relatively low velocities, whereas concentrations of boulders would produce higher velocities. This is consistent with results obtained from other seismic investigations in similar geological settings; however, this interpretation is not a unique one. The velocities obtained during this investigation in the Santa Ana River are reasonable and are comparable to those we have measured in similar river deposits and alluvial fans.

The velocities measured at the top of Layer 4, bedrock, are apparent velocities. Apparent velocities differ from true velocities if the layers are not horizontal. However, the true velocity of the bedrock can be approximated by averaging the apparent velocities obtained from the shot points at the ends of the line and/or from the in-line offset shot points.

The three survey lines across the River (Figures 3, 4 and 5) are interpreted to indicate that in these areas, the bedrock surface has a generally uniform slope toward the west side of the channel. Interpreted depth to bedrock, measured at the cross-over points on the time-distance graphs, ranges from 50 to 60 feet near the east end to 75 to 100 feet near the west end of the lines. This sloping channel bottom appears to make a fairly sharp join with the steep western canyon wall (see Figures 3 and 5). A "U-shaped" bedrock surface with the trough of the "U" in the center of the channel is not a likely interpretation based on the available seismic data because the data do not indicate the predicted sharp changes in velocity across the channel. The survey line oriented essentially parallel to the channel (Figure 6) is interpreted as showing the bedrock surface with a gentle slope toward the southwest (toward the mouth of the river). Depth to

bedrock ranges from 70 to 80 feet along the survey line. The two in-line offset shots were valuable in verifying the velocity of the bedrock at greater than 10,000 ft/sec and in supporting the interpretation that the 6,000-8,500 ft/sec velocity is saturated alluvium composed of boulders and cobbles.

Reference Cited

1. Leighton and Associates, Inc., 1984, Preliminary interpretations of a seismic refraction investigation at the proposed Mentone Dam site: 5 p. Project No. 1840081-01, dated April 16, 1984.

If you have any questions regarding this report, please do not hesitate to contact this office. We appreciate this opportunity to be of service.

Respectfully submitted,

LEIGHTON AND ASSOCIATES, INC.

David Cummings, GP 153
Director of Geophysics

GDC/sdb/cas

Distribution: (3) Addressee

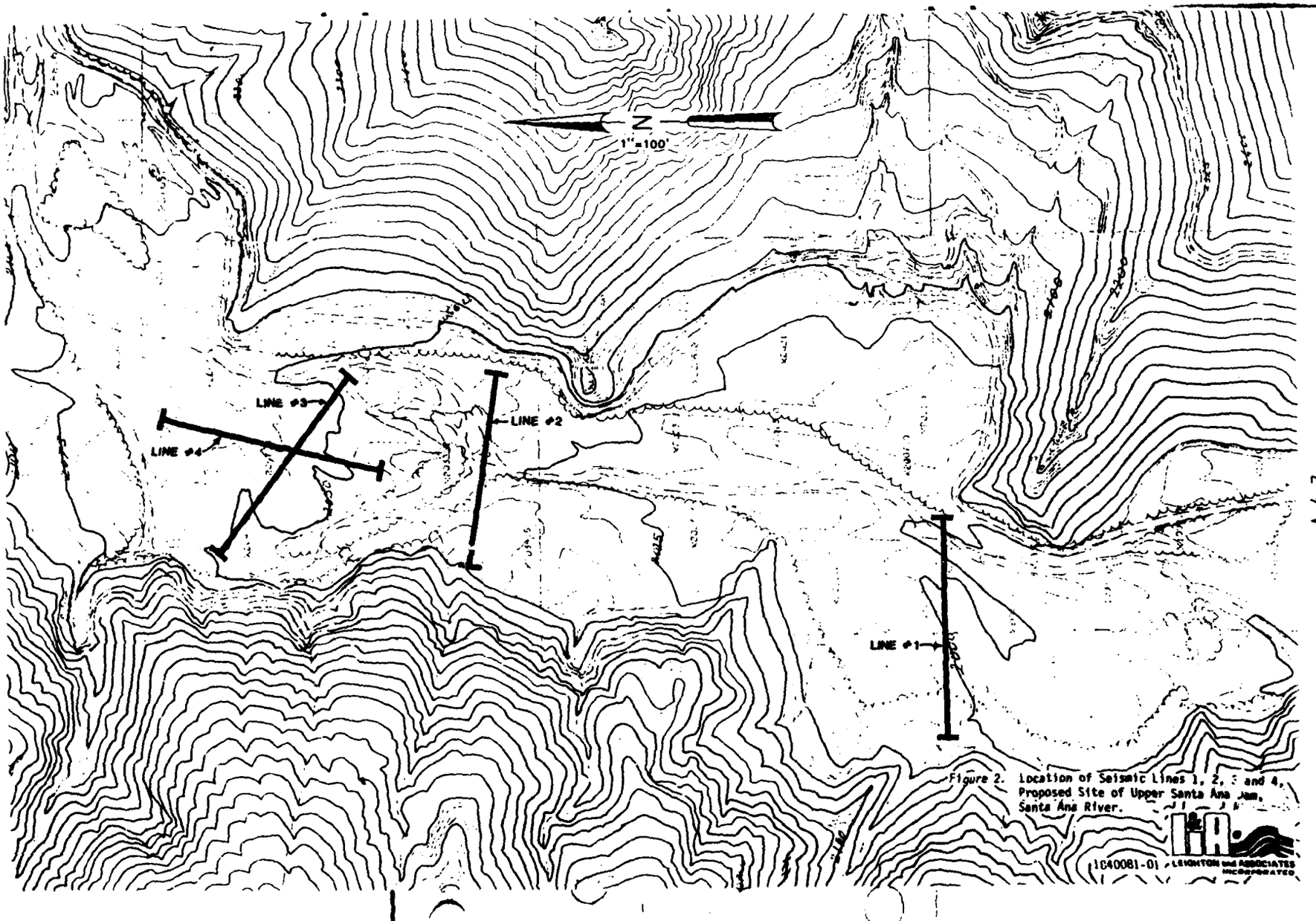
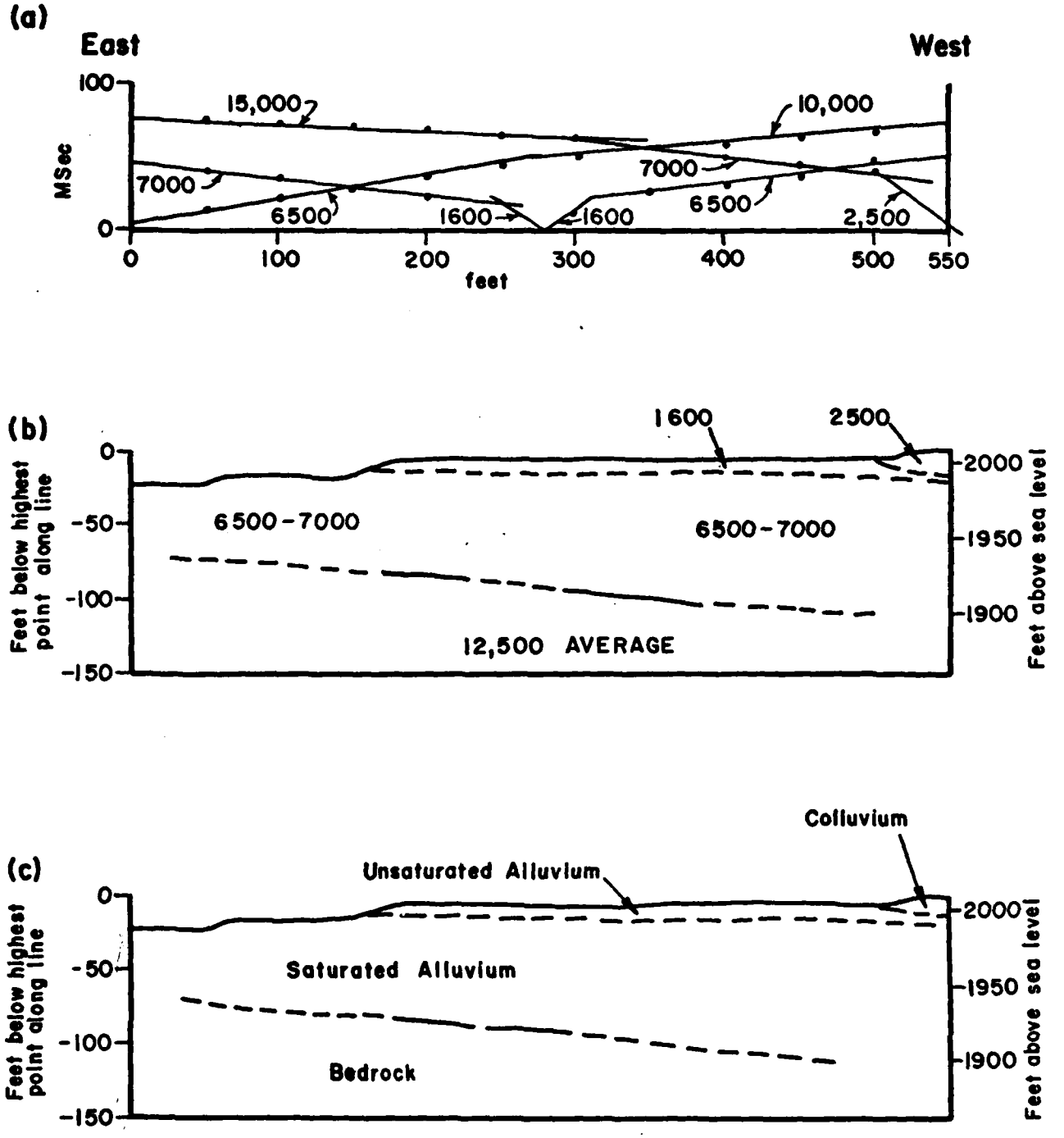


Figure 2. Location of Seismic Lines 1, 2, 3 and 4, Proposed Site of Upper Santa Ana Dam, Santa Ana River.

(1040081-01)

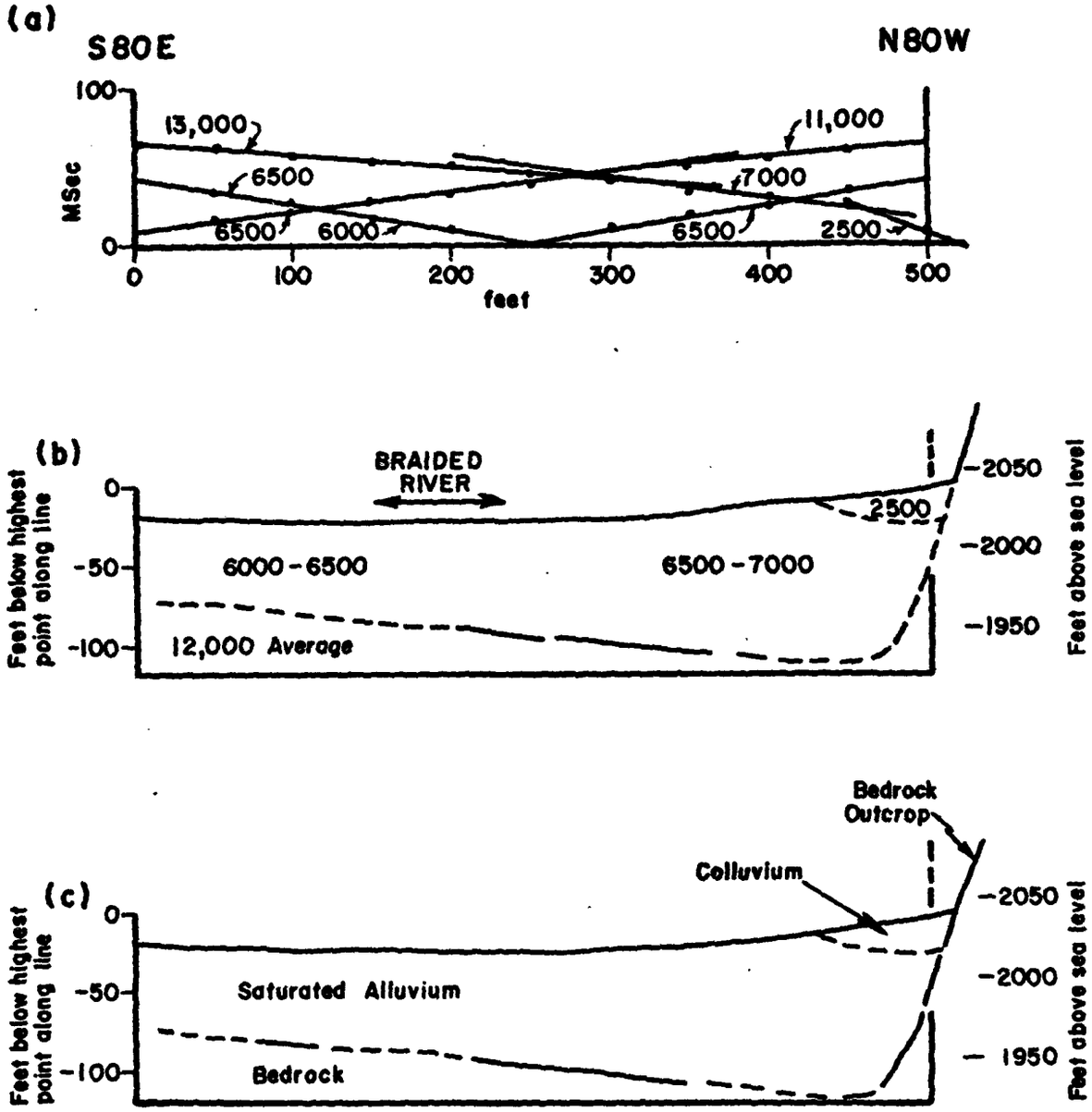
LEIGHTON and ASSOCIATES INCORPORATED



Note: Cross-section plotted as viewed downstream.

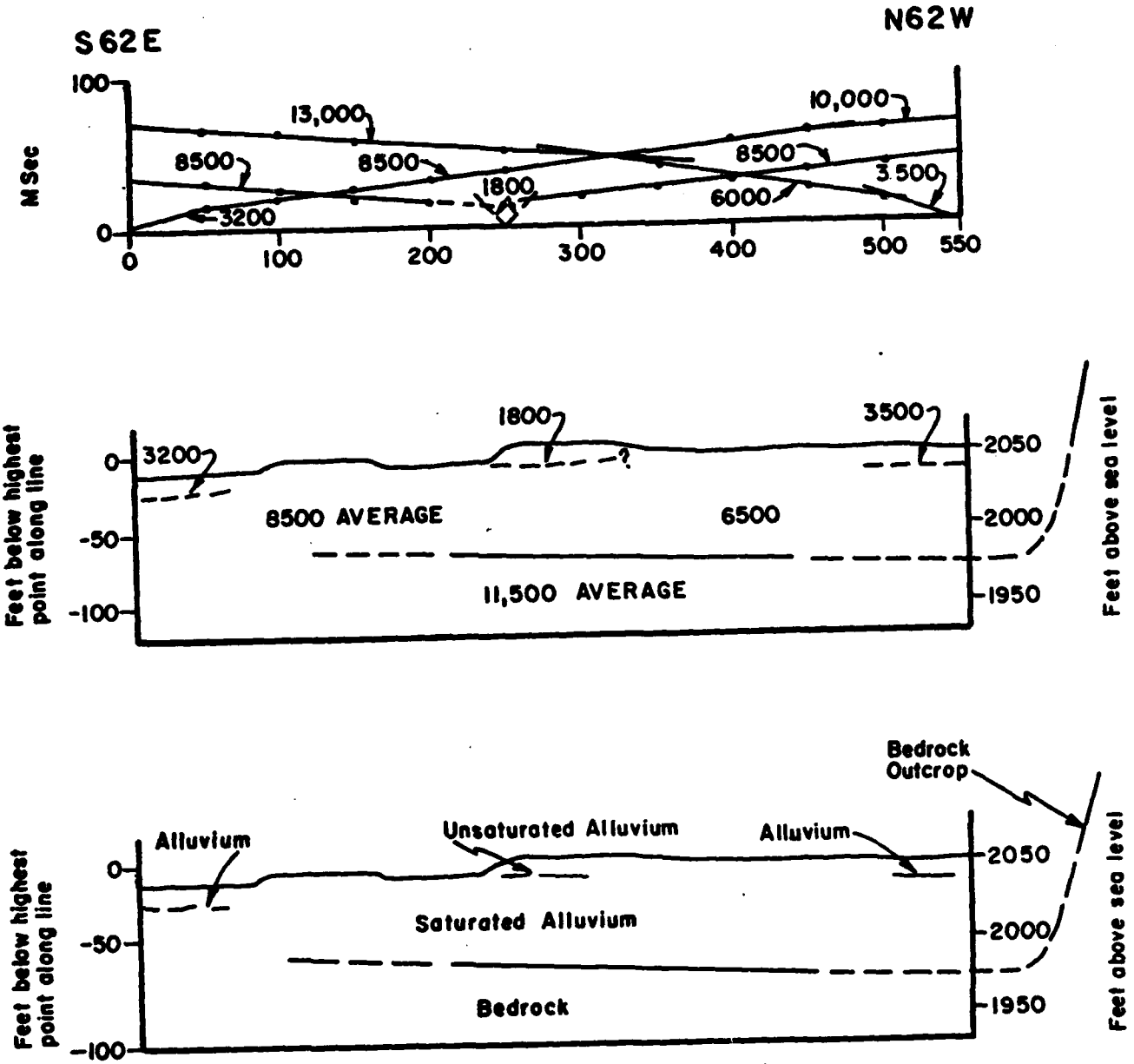
Figure 3. Seismic Line 1. (a) Time-distance graph of corrected field data. (b) Geophysical interpretation of data. (c) Inferred geologic interpretation of geophysical data and from geologic field observation.

L&A
LEIGHTON and ASSOCIATES
INCORPORATED



Note: Cross-section plotted as viewed downstream.

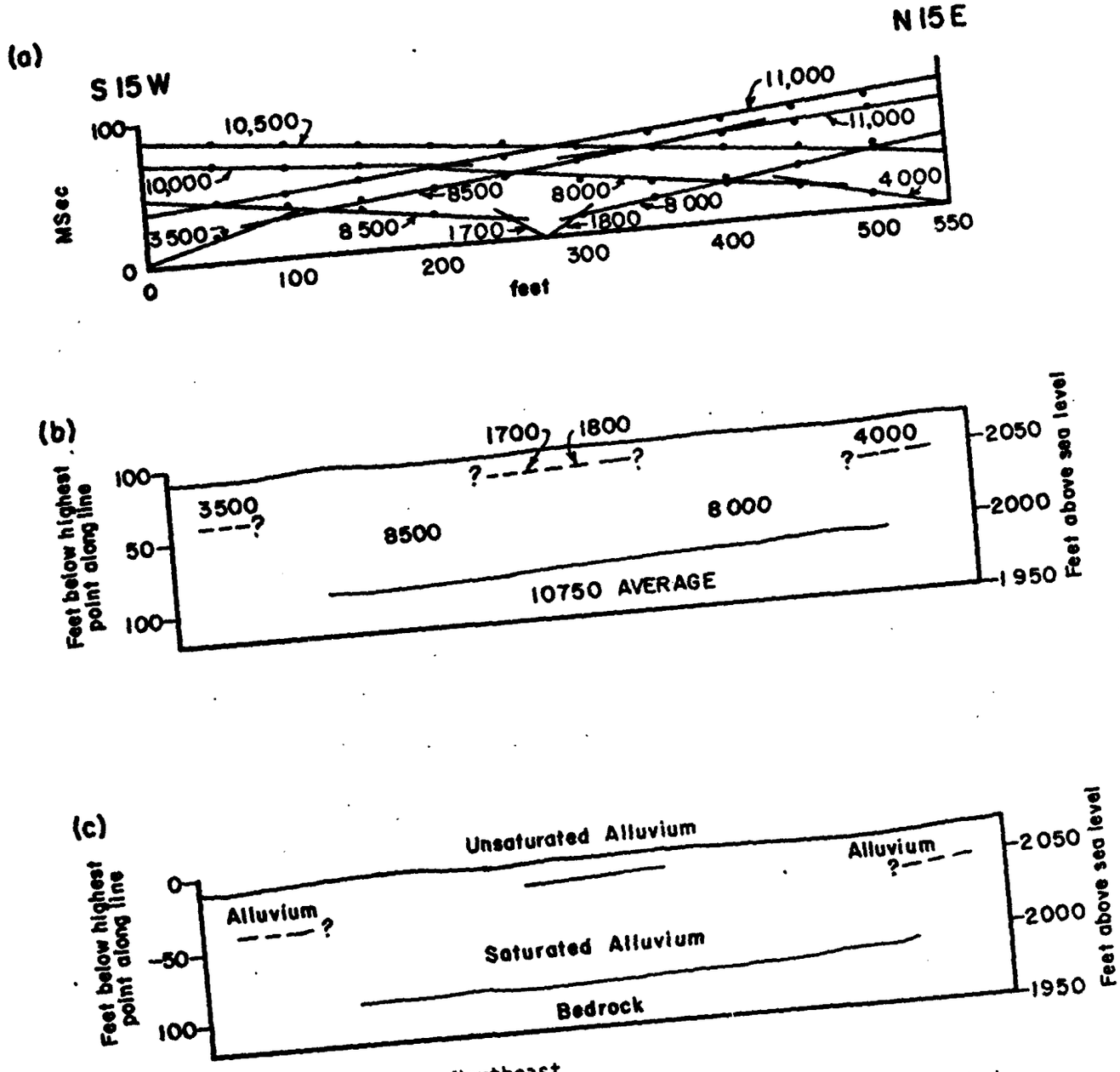
Figure 4. Seismic Line 2. (a) Time-distance graph of corrected field data. (b) Geophysical interpretation of data. (c) Inferred geologic interpretation of geophysical data and from geologic field observation.



Note: Cross-section plotted as viewed downstream.

Figure 5. Seismic Line 3. (a) Time-distance graph of corrected field data; velocities in feet/second. (b) Geophysical interpretation of data; velocities in feet/second. (c) Inferred geologic interpretation of geophysical data and from geologic observation.

LEIGHTON and ASSOCIATES INCORPORATED



Note: Upstream to Northeast

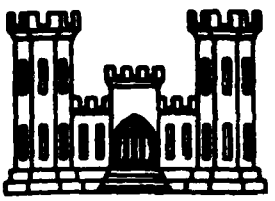
Figure 6. Seismic Line 4. (a) Time-distance graph of corrected field data; velocities in feet/second. (b) Geophysical interpretation of data; velocities in feet/second. (c) Inferred geologic interpretation of geophysical data and from geologic observation.

A GEOTECHNICAL

ATTACHMENT
A-5

South Pacific Division Laboratory
Report of Soil Tests

DEPARTMENT OF THE ARMY
SOUTH PACIFIC DIVISION, CORPS OF ENGINEERS
LABORATORY



REPORT OF SOIL TESTS
UPPER SANTA ANA RIVER

AUGUST 1985

SAUSALITO, CALIFORNIA

REPORT

OF

SOIL TESTS

UPPER SANTA ANA RIVER

AUGUST 1985

AUTHORIZATION

1. Results of tests reported herein were requested by the Los Angeles District in laboratory request No. E86-86-0066 dated 9 April 1985.

SAMPLES

2. Four disturbed samples in steel drums were received on 28 March 1985. Identification of samples is on the Soil Test Result Summary Plate 1.

TESTING PROGRAM

3. The program was in accordance with the test request and included sieve analyses, specific gravity, triaxial shear.

TEST METHODS

4. Grain-size Analysis, Triaxial Compression, Specific Gravity. Testing methods conformed to the procedures described in Engineer Manual, EM 1110-2-1906, "Laboratory Soil Testing," 30 November 1970.

RESULTS

5. Results of tests are shown on the following plates:

Subject	Plate No.
Soil Test Result Summary	1
Triaxial Compression Test Report	2 - 5

U.S. ARMY ENGINEER DIVISION LABORATORY -- SOUTH PACIFIC DIVISION

SOIL TEST RESULT SUMMARY

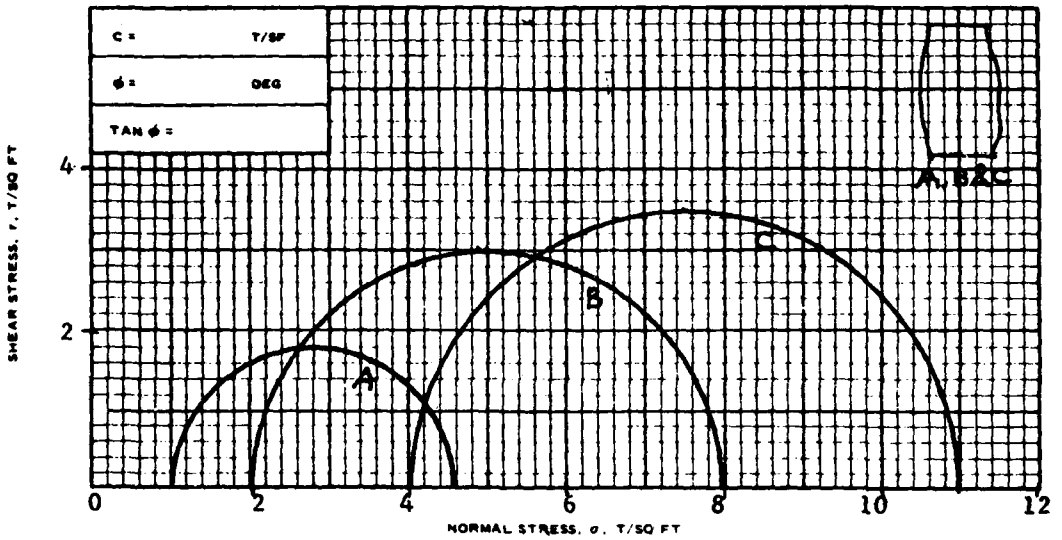
PROJECT Upper Santa Ana River

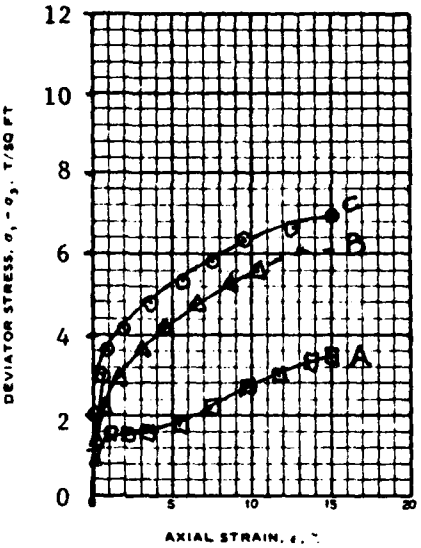
DATE August 1985

Division Serial No:	Hole No.	Field Sample No.	Depth Or Elevation From	Depth Or Elevation To	Laboratory Descriptive Classification	Mechanical Analysis-% Finer: Gravel 6	3	2	1½	1	3/4	1/2	3/8	Sand #4	#10	#40	#60	#100	Fines #200	Specific Gravity +4	Specific Gravity -4	Field Moist. %
Composite "A"																						
91953	TP1-3				Sandy Gravel(GL	100		80		65												
						92		75		59		50	45	36	29	8	4	2	1	2.67	2.65	
91955	TP3-1																					
Composite "B"																						
91954	TP2-3				Sandy Gravel(GP	100		87		78												
						94		83		75		71	69	65	60	18	8	4	2	2.68	2.66	
91956	TP3-2																					

SPD Form 66A
1 May 83

Plate 1





SPECIMEN NO.			A	B	C
INITIAL	WATER CONTENT, %	w_o	3.0	3.0	2.9
	DRY DENSITY LB/ CU FT	γd_o	122.1	122.0	122.1
	SATURATION, %	s_o	22	22	22
	VOID RATIO	e_o	0.360	0.361	0.360
BEFORE SHEAR	WATER CONTENT, %	w_c	13.4	12.9	12.9
	DRY DENSITY LB/CU FT	γd_c	122.4	123.0	123.5
	SATURATION, %	s_c	100	98	100
	VOID RATIO	e_c	0.356	0.349	0.344
	FINAL BACK PRESSURE, T/SQ FT	u_b	5.04	5.04	5.04
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	1.00	2.00	4.00
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	3.57	6.00	7.06
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	378	-	398
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$	-	-	-
INITIAL DIAMETER, IN.		D_o	12.0	12.0	12.0
INITIAL HEIGHT, IN.		H_o	27.65	27.65	27.65

CONTROLLED- Strain TEST

DESCRIPTION OF SPECIMENS Sandy Gravel (GP)

LL	PL	PI NP	Gs 2.66	TYPE OF SPECIMEN Remolded	TYPE OF TEST $\bar{R}$

REMARKS: Specimen B membrane punctured after 10% strain

Remolded to 122 pcf composite

Div. No.91953 & 91955

PROJECT Upper Santa Ana River

BORING NO. TP1-3 & TP3-1 SAMPLE NO. Comp "A"

DEPTH/ELEV

LABORATORY SPDL DATE June 1985

TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089
REV JUNE 1970

plate 2

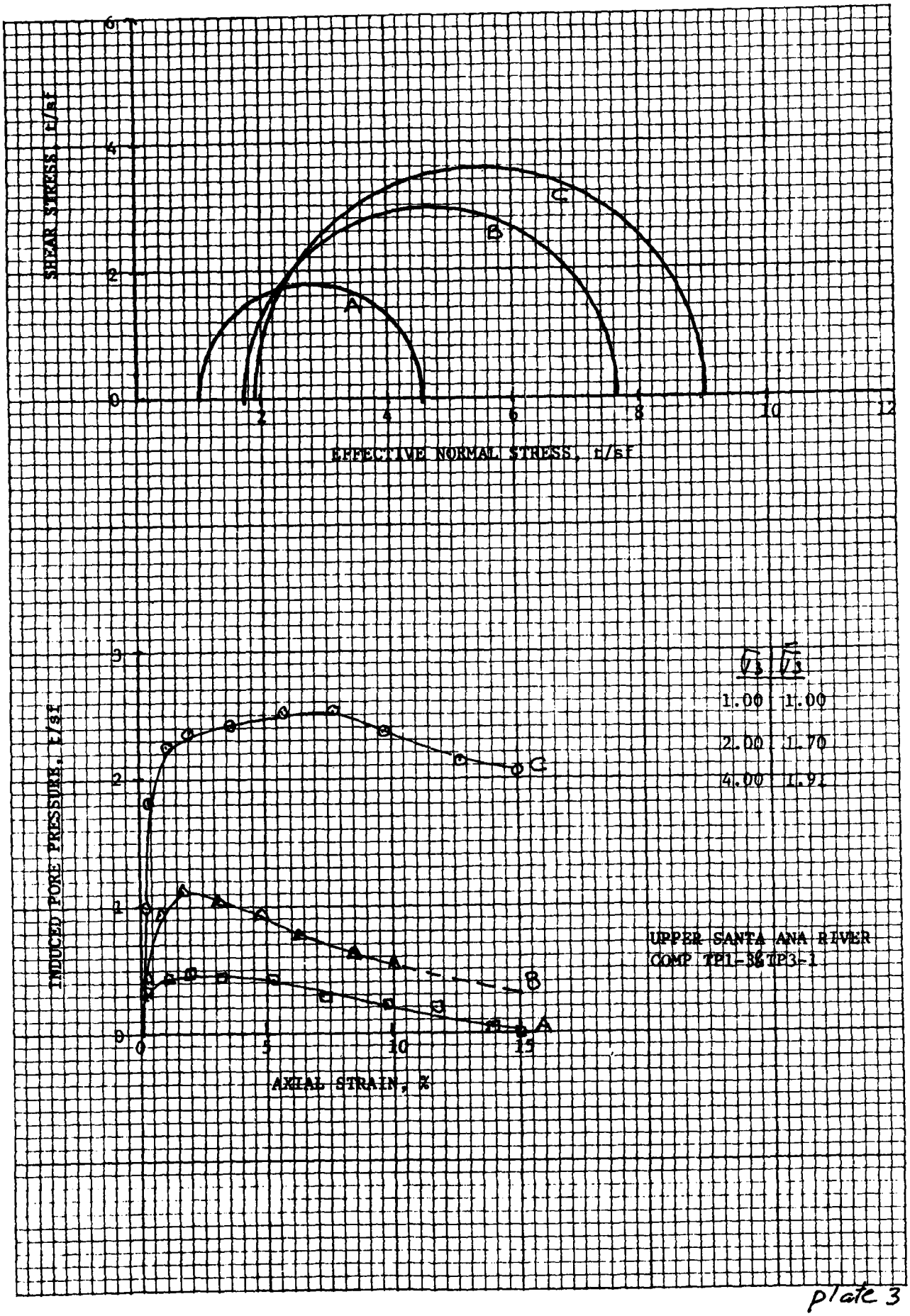
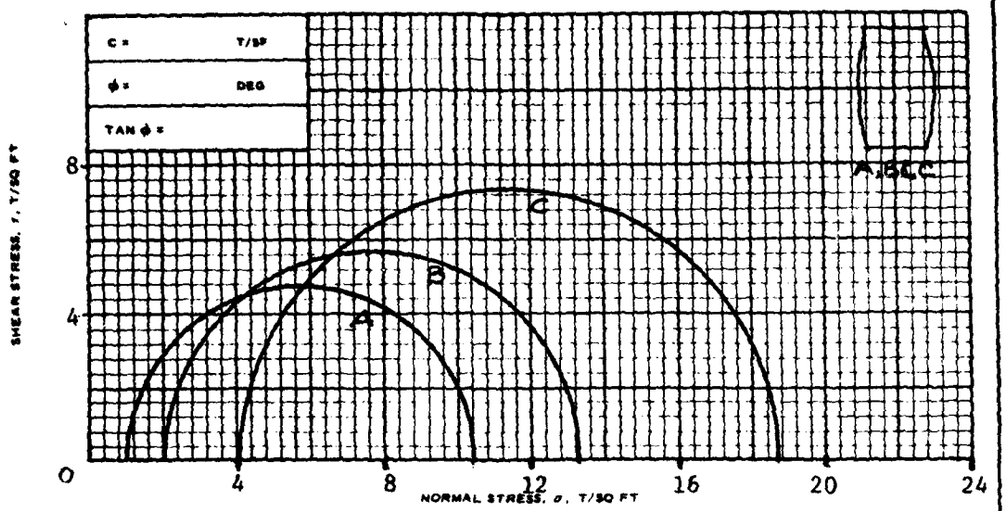
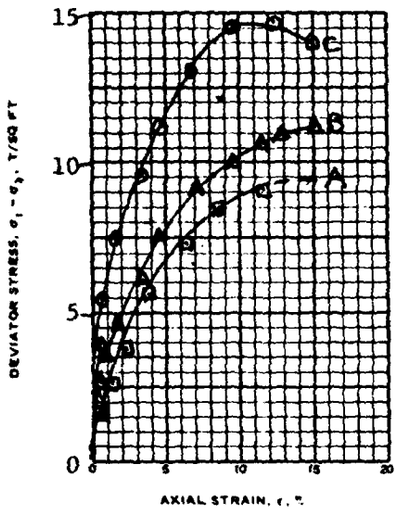

SHEAR STRESS, t/sf
EFFECTIVE NORMAL STRESS, t/sf
A
B
C
INDUCED PORE PRESSURE, t/sf
AXIAL STRAIN, %
$\bar{\sigma}_3$ / $\bar{\sigma}_1$
1.00 / 1.00
2.00 / 1.70
4.00 / 1.91
UPPER SANTA ANA RIVER
COMP TP1-3 & TP3-1

plate 3





SPECIMEN NO.			A	B	C
INITIAL	WATER CONTENT, %	w_o	2.6	2.6	2.5
	DRY DENSITY LB/CU FT	γ_{d_o}	113.4	113.3	113.4
	SATURATION, %	S_o	15	15	14
	VOID RATIO	e_o	0.470	0.470	0.469
BEFORE SHEAR	WATER CONTENT, %	w_c	17.4	17.0	16.9
	DRY DENSITY LB/CU FT	γ_{d_c}	113.8	114.2	114.7
	SATURATION, %	S_c	100	99	100
	VOID RATIO	e_c	0.464	0.459	0.453
	FINAL BACK PRESSURE, T/SQ FT	u_b	5.04	5.04	5.04
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	1.00	2.00	4.00
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	9.40	11.32	14.73
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	–	392	370
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$	–	–	14.07
INITIAL DIAMETER, IN.		D_o	12.0	12.0	12.0
INITIAL HEIGHT, IN.		H_o	27.65	27.65	27.65

CONTROLLED- Strain TEST

DESCRIPTION OF SPECIMENS Sandy Gravel (GP)

LL	PL	PI	G_s
		NP	2.67

TYPE OF SPECIMEN Remolded | TYPE OF TEST $\bar{R}$

REMARKS: Specimens remolded to 113 pcf.
Speciman A membrane punctured at 12%.

PROJECT Upper Santa Ana River

BORING NO. TP2-3 & TP3-2 | SAMPLE NO. Comp. "B"

DEPTH/ELEV

LABORATORY SPDL | DATE July 1985

Div. No. 91954 & 91956

TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089
REV JUNE 1970

plate 4

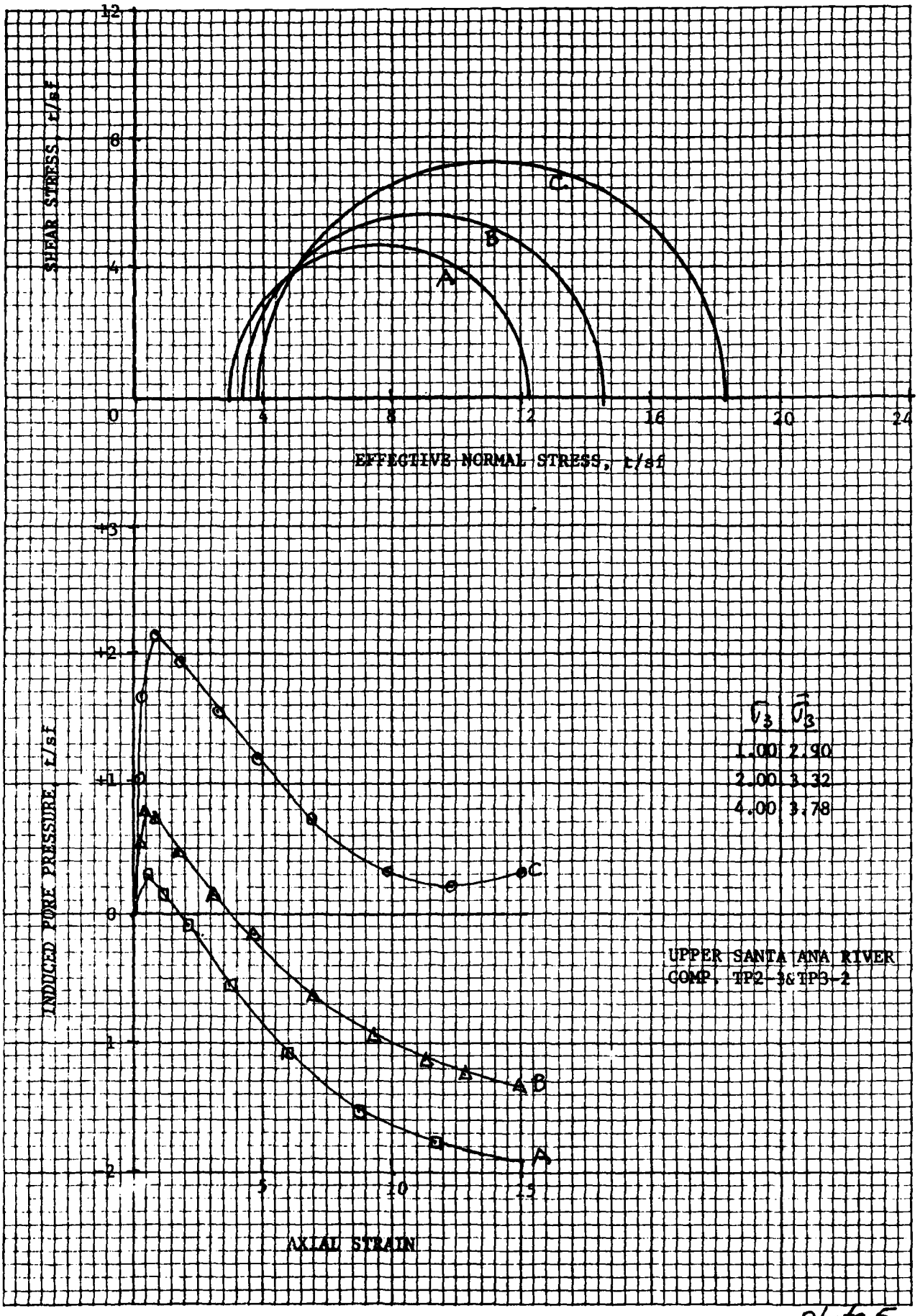
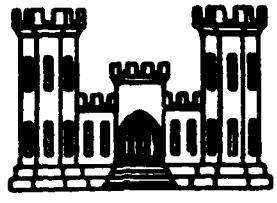

SHEAR STRESS, t/sf
12
8
4
0
4
8
12
16
20
24
C
B
A
EFFECTIVE NORMAL STRESS, t/sf
INDUCED PORE PRESSURE, t/sf
+3
+2
+1
0
1
2
5
10
15
AXIAL STRAIN
$\bar{\sigma}_3$ | $\bar{\sigma}_3$
1.00 | 2.90
2.00 | 3.32
4.00 | 3.78
UPPER SANTA ANA RIVER
COMP. TP2-3&TP3-2

plate 5

DEPARTMENT OF THE ARMY
SOUTH PACIFIC DIVISION, CORPS OF ENGINEERS
LABORATORY



REPORT OF SOIL TESTS

UPPER SANTA ANA RIVER, CA

UPSTREAM CORE BORROW SITE

NOVEMBER 1985

SAUSALITO, CALIFORNIA

REPORT
OF
SOIL TESTS

UPPER SANTA ANA RIVER, CA
UPSTREAM CORE BORROW SITE

NOVEMBER 1985

AUTHORIZATION

1. Results of tests reported herein were requested by the Los Angeles District in laboratory request No. E86-85-0081 dated 3 June 1985.

SAMPLES

2. Ten disturbed samples contained in sacks were received 6 May 1985. Identification of samples is shown on the Soil Test Result Summary, Plate I.

TESTING PROGRAM

3. The program was in general accordance with the test request, and included sieve analysis, Atterberg limits, specific gravity, compaction, consolidation, permeability, triaxial shear, direct shear and dispersion tests.

TEST METHODS

4. a. Grain-size Analysis, Atterberg Limits, Specific Gravity, Triaxial and Direct Shear, Permeability, Consolidation, and Compaction. Testing conformed to the procedures described in Engineer Manual, EM 1110-2-1906, "Laboratory Soil Testing," 30 November 1970.

b. Classification. The soil was classified in accordance with "The Unified Soil Classification System," TM No. 3-357, Appendix A, April, 1960.

c. SCS Laboratory Dispersion Test. Test procedures conformed to Appendix A of "Piping in Earth Dams of Dispersive Clay," Sherard, Decker and Ryker.

RESULTS

5. Results of tests are shown below and on the following plates:

SUBJECT	PLATE NO.
Soil Test Result Summary	1
Compaction Test Report	2-6
Direct Shear Test Report	7-11
Triaxial Compression Test Report	12-17
Permeability	18-19
Gradation Curves - SCS Dispersion Test	20-23
Consolidation Test Report	24-35

COMMENTS

6. Dispersion Testing. Pinhole test was attempted but could not be completed because of collapse of the pinhole which caused a blockage and prevented the flow of water. The SCS double hydrometer and the crumb tests were substituted for the pinhole test. The crumb test results showed all samples to be nondispersive. The SCS test is applicable only to samples which have more than 12% passing the 0.005 mm size and have a PI greater than 4. Sample 92447 has only 7.5% passing the 0.005 mm size. The percent dispersion ranges are: 0-35% Nondispersive; 35-50% Intermediate; 50-100% Dispersive.

U.S. ARMY ENGINEER DIVISION LABORATORY -- SOUTH PACIFIC DIVISION

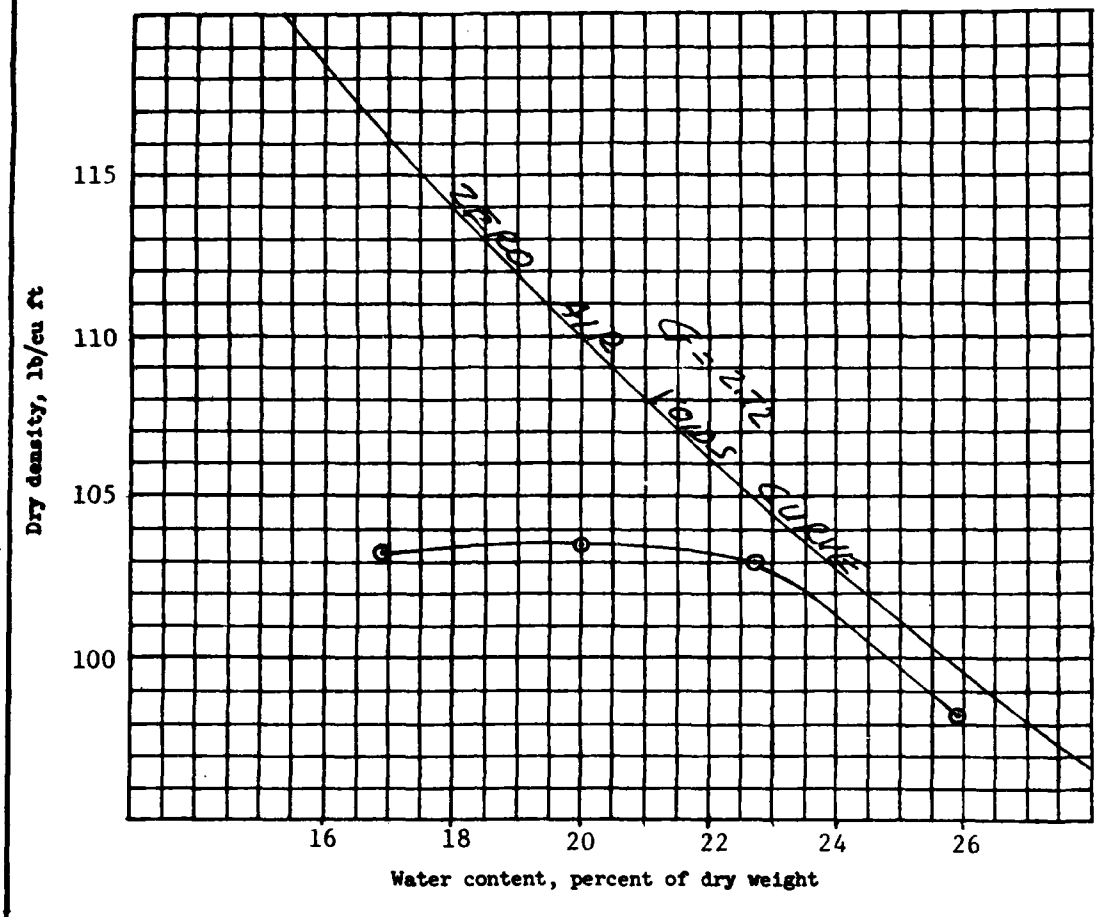
SOIL TEST RESULT SUMMARY

PROJECT UPPER SANTA ANA RIVER - U/S CORE BORROW **DATE** July 1985

Division Serial No.	Hole No.	Field Sample No.	Depth Or Elevation		Laboratory Descriptive Classification	Mechanical Analysis-% Finer												Liquid Limit	Plasticity Index	Field Moist. %
						Gravel					Sand					Fines				
			From	To		3 2	1½ 1	3/4	1/2	3/8	#4	#10	#40	#60	#100	#200				
92444	TH-1	6	18	21	Silty Sandy Gravel (GM)		100 95	90	81	75	61	59	42	38	35	30		47	18	
92445	"	7	27	30	Sandy Silty Gravel (GM)	100 98	96 93	90	84	80	71	69	66	65	62	47		33	9	
92446	TH-5	11	30	33	Sandy Gravelly Clay (CL)		- 100	99	95	89	73	71	68	67	66	60		35	15	
92447	"	16	47.5	50	Clayey Sand (SC-SM)		100 99	99	98	98	95	94	92	88	70	44		27	7	
92448	"	19	55.5	57	Gravelly Silty Clay (CL)			100	99	98	86	84	83	82	81	77		37	14	
92449	TH-6	4	8	11	Silty Sand(SM)							100	95	81	58	35			NP	
92450	TH-7	9	20	23	Sandy Gravelly Clay (CL)			100	98	93	73	69	63	61	58	51		38	22	
92451	"	12	27	28	Sandy Clay (CL)				100	99	88	84	75	72	68	57		33	12	
92452	TH-9	8	18.5	20	Sandy Clay (CL)				100	99	91	90	86	83	76	60		35	16	
92453	TH-10	16	30	34	Sandy Clay (CL)						100	99	97	96	94	80		41	20	

Plate 1

SPD Form 66A
1 May 83



Modified compaction test

25 blows per each of 5 layers, with 10 lb rammer and

18 inch drop. 4 inch diameter mold

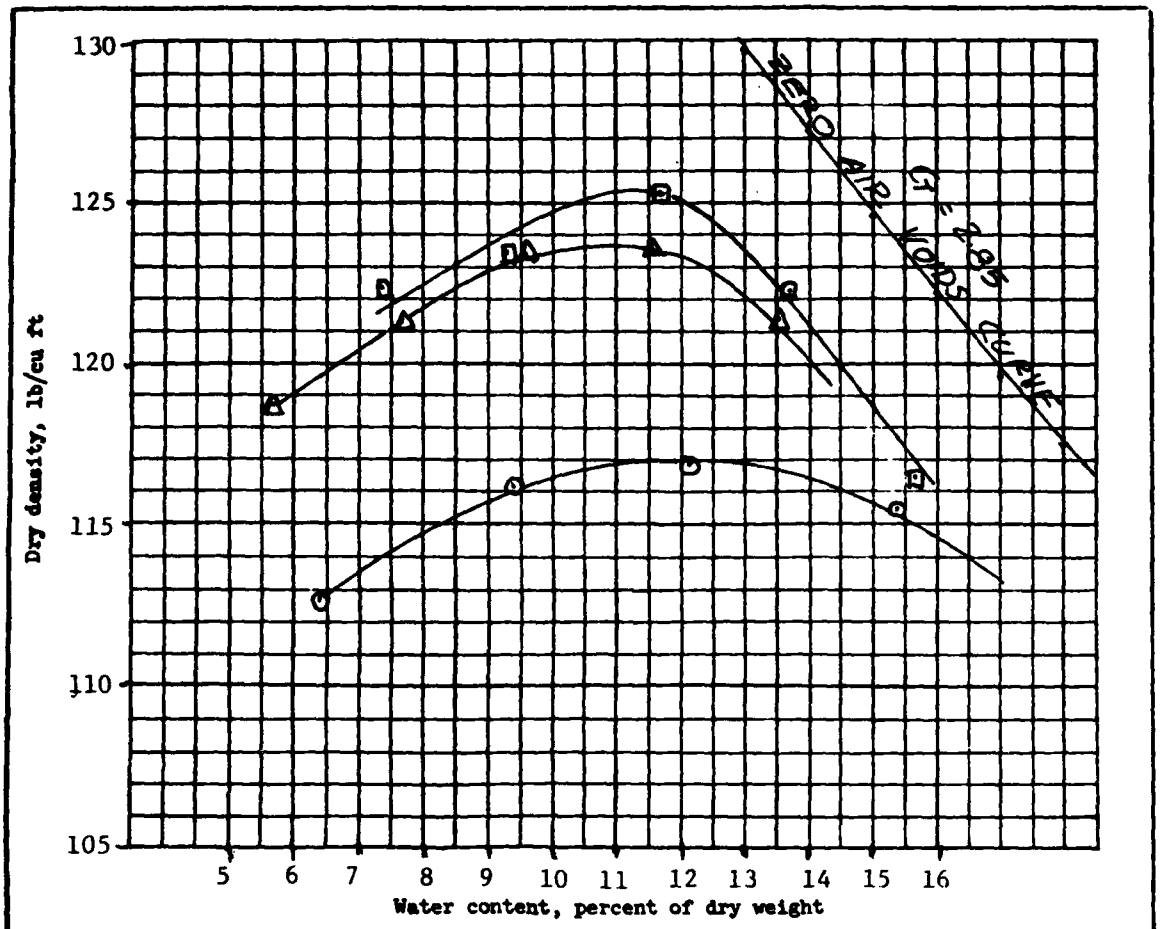
Sample No.	Elev or Depth	Classification	G	LL	PL	% > No. 4	% > 3/4 in.
FS 6	18-27	Silty Sandy Gravel (GM) (Silty Sand) (SM)	2.72	47	29	39	10

Sample No.	92444			
Natural water content, percent				
Optimum water content, percent	20.0			
Max dry density, lb/cu ft	103.6			

Remarks	Project Upper Santa Ana River
	U/S Core Borrow
	Area Seven Oaks
	Boring No. TH-1 Date July 1985
	COMPACTION TEST REPORT

ENG FORM 1 JUN 65 2091 PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT)

plate 2



Modified compaction test

25 blows per each of 5 layers, with 10 lb rammer and

18 inch drop. 4 inch diameter mold

Sample No.	Elev or Depth	Classification	G	LL	PL	% > No. 4	% > 3/4 in.
7	27-30	Sandy Silty Gravel (GM)	2.81	33	24	39	10
11	30-33	Sandy Gravelly Clay (CL)	2.79	35	20	27	1
16	47.5-50	Clayey Sand (SC-SM)	2.85	27	20	5	1

Sample No.	92445	92446	92447	
Natural water content, percent	⊙	△	□	
Optimum water content, percent	12.1	11.0	11.3	
Max dry density, lb/cu ft	117.0	123.7	125.4	

Remarks		
Div.No.	Hole	FS
92445	TH-1	7
92446	TH-5	11
92447	"	16

Project Upper Santa Ana River

U/S Core Borrow

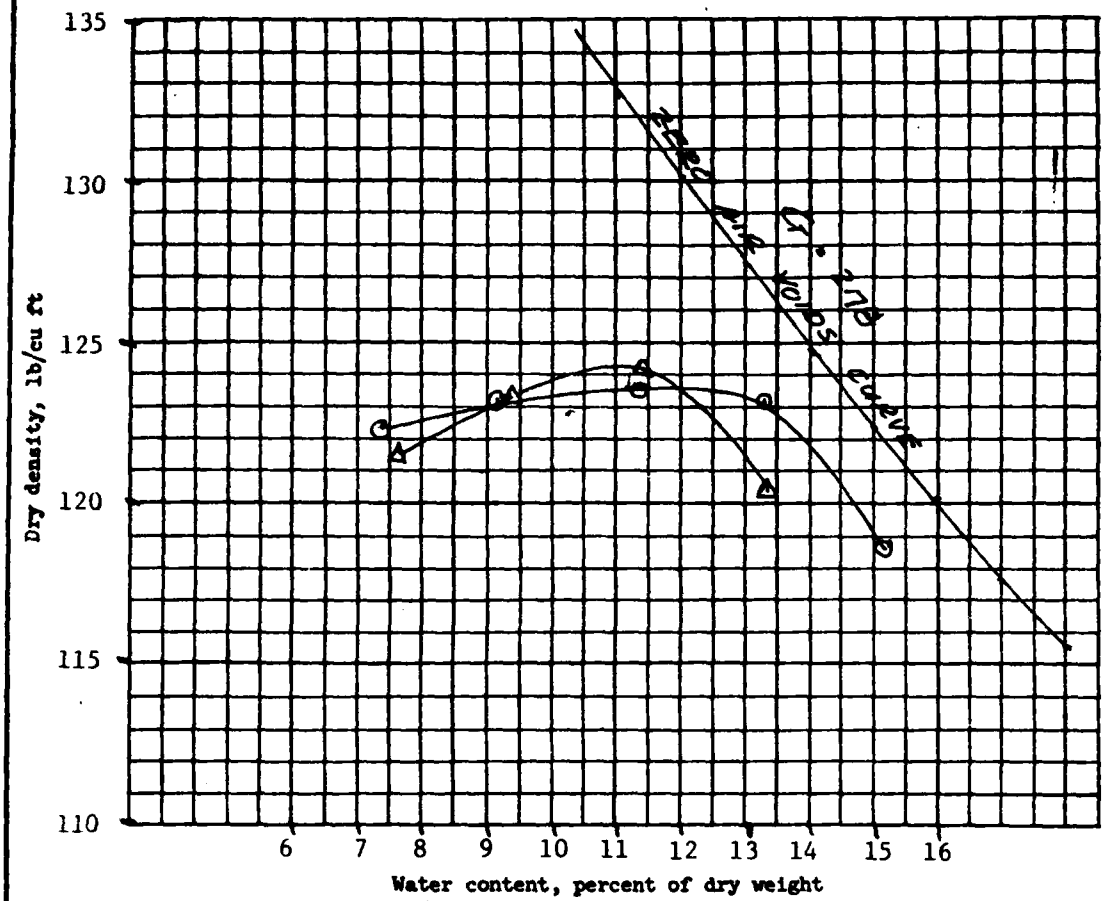
Area

Boring No. TH-1, -5 Date July 1985

COMPACTION TEST REPORT

ENG FORM 1 JUN 65 2091 PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT)

plate 3



Modified compaction test

25 blows per each of 5 layers, with 10 lb rammer and

18 inch drop. 4 inch diameter mold

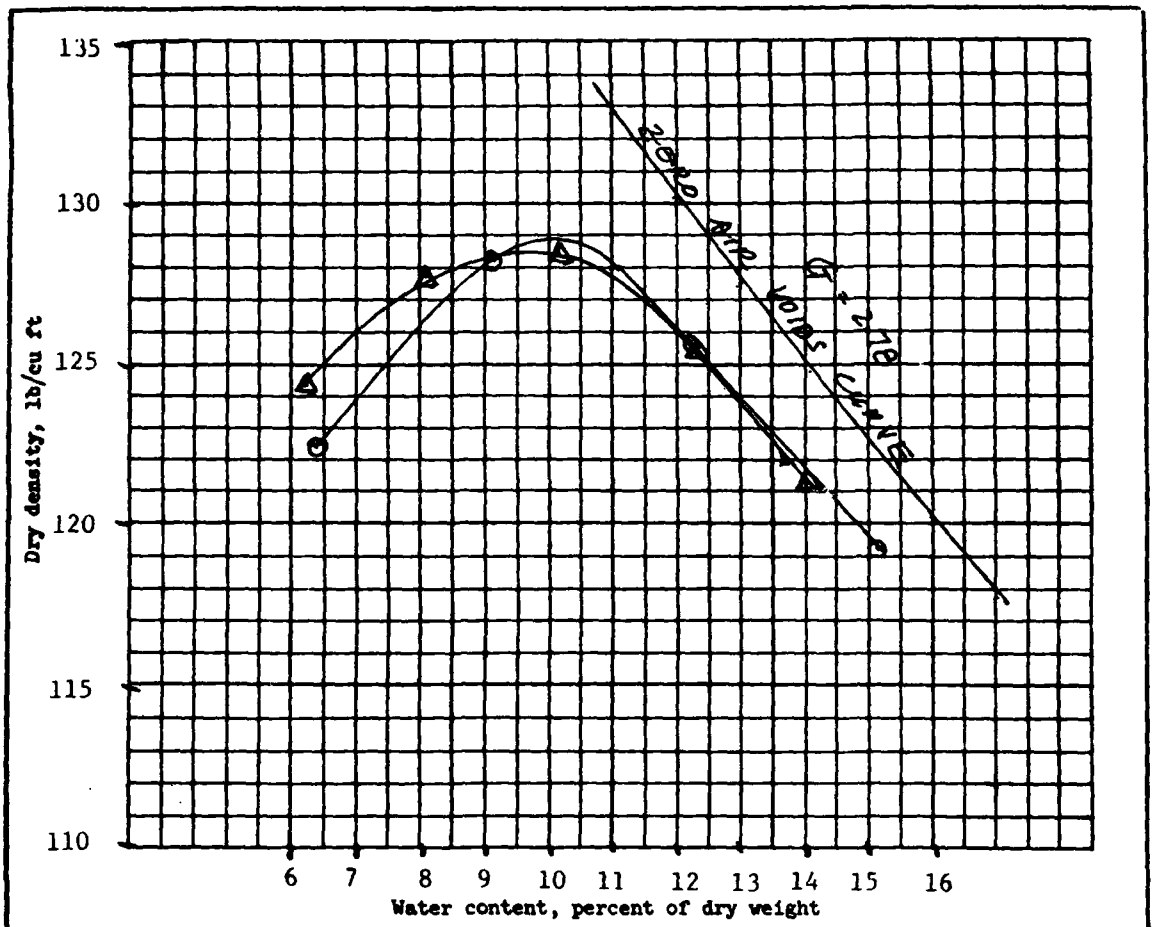
Sample No.	Elev or Depth	Classification	G	LL	PL	% > No. 4	% > 3/4 in.
19	55.5-57	Gravelly Silty Clay (CL)	2.76	37	23	14	0
4	8 - 11	Silty Sand (SM)	2.78		NP	0	0

Sample No.	92448	92449		
Natural water content, percent	⊙	△		
Optimum water content, percent	12.0	11.0		
Max dry density, lb/cu ft	123.7	124.3		

Remarks			Project Upper Santa Ana River	
Div.No.	Hole	FS	U/S Core Borrow	
92448	TH-5	19	Area	
92449	TH-6	4	Boring No. TH-5 & TH-6	Date July 1985
			COMPACTION TEST REPORT	

ENG FORM 1 JUN 65 2091 PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT)

plate 4



Modified compaction test

25 blows per each of 5 layers, with 10 lb rammer and

18 inch drop. 4 inch diameter mold

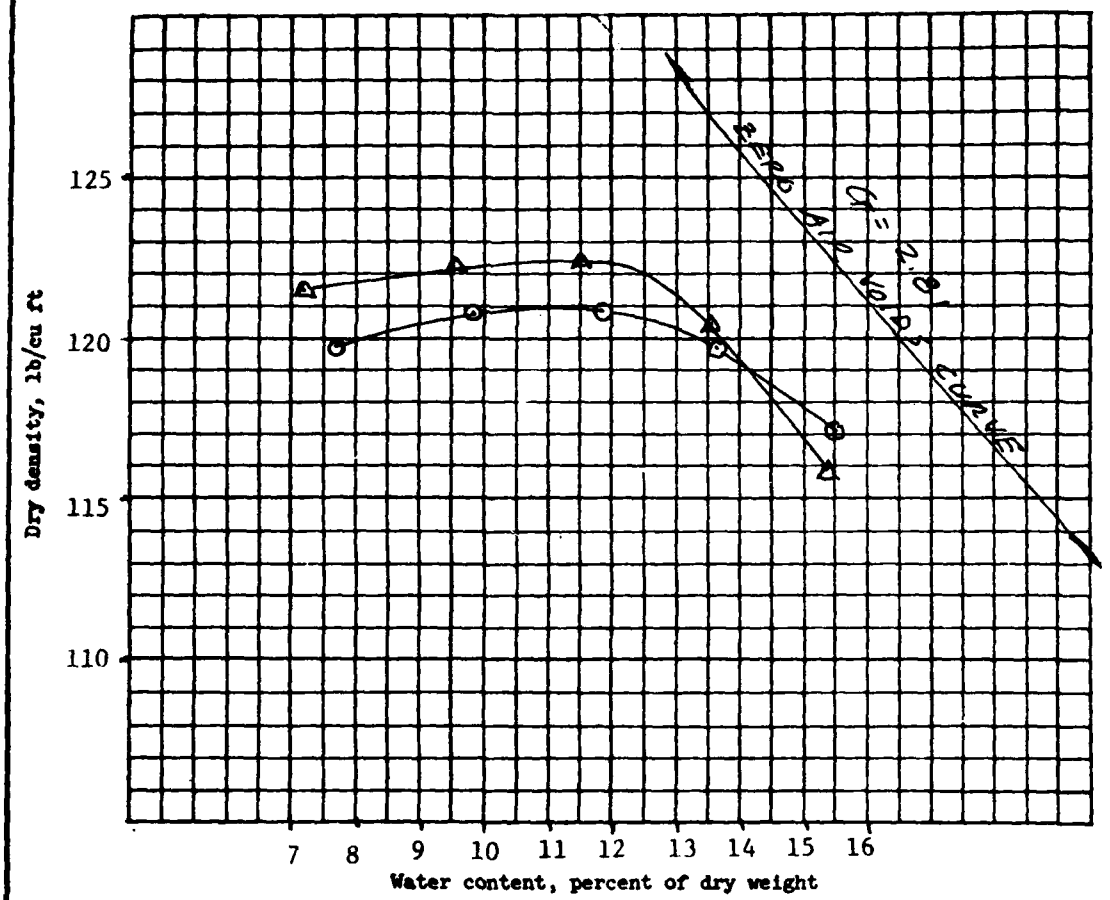
Sample No.	Elev or Depth	Classification	G	LL	PL	% > No. 4	% > 3/4 in.
9	20-23	Sandy Gravelly Clay (CL)	2.77	38	16	27	0
12	27-28	Sandy Clay (CL)	2.78	33	12	12	0

Sample No.	92450	92451		
Natural water content, percent	○	△		
Optimum water content, percent	10.1	9.8		
Max dry density, lb/cu ft	128.9	128.5		

Remarks		Project Upper Santa Ana River
Div.No.	FS	U/S Core Borrow
92450	9	Area
92451	12	Boring No. TH-7 Date August 1985
		COMPACTION TEST REPORT

ENG FORM 1 JUN 65 2091 PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT)

plate 5



Modified compaction test

25 blows per each of 5 layers, with 10 lb rammer and

18 inch drop. 4 inch diameter mold

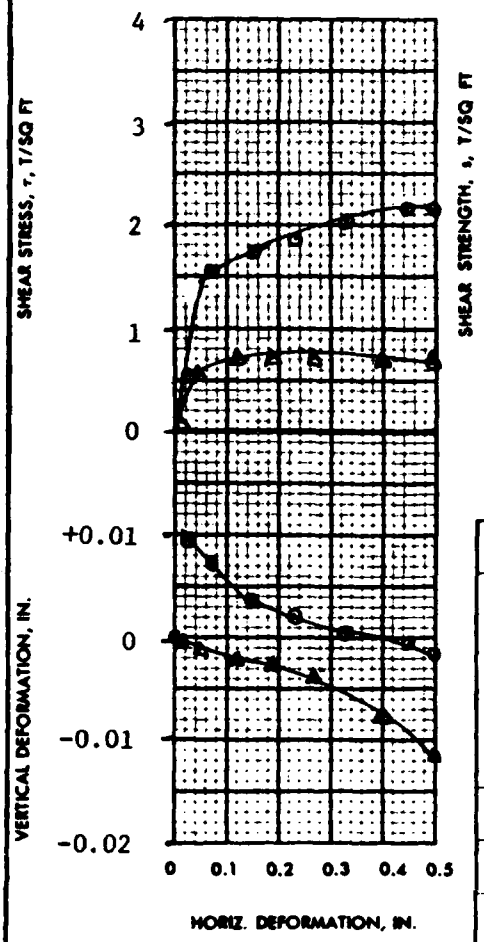
Sample No.	Elev or Depth	Classification	G	LL	PL	% > No. 4	% > 3/4 in.
8	18.5-20	Sandy Clay (CL)	2.81	35	19	9	0
16	30-34	Sandy Clay (CL)	2.25	41	21	0	0

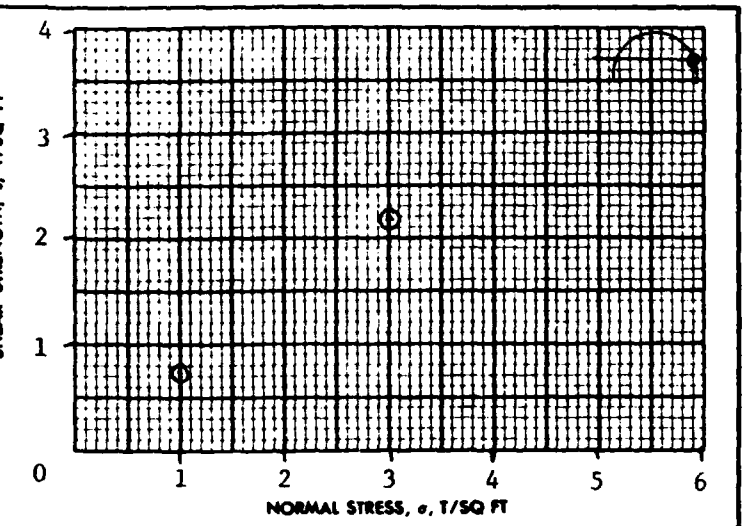
Sample No.	92452	92453		
Natural water content, percent				
Optimum water content, percent	11.2	11.5		
Max dry density, lb/cu ft	121.0	122.5		

Remarks			Project Upper Santa Ana River	
Div. No.	Hole	FS		
92452	TH-9	8	Area U/S Core Borrow	
92453	TH-10	16	Boring No. TH-9 & TH-10	Date August 1985
			COMPACTION TEST REPORT	

ENG FORM 1 JUN 65 2091 PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT)

plate 6





SHEAR STRENGTH PARAMETERS

ϕ' = _____

TAN ϕ' = _____

c' = _____ T/SQ FT

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

TEST NO.			A	B		
INITIAL	WATER CONTENT	w_o	22.2 %	22.1 %	%	%
	VOID RATIO	e_o	0.823	0.827		
	SATURATION	S_o	73 %	73 %	%	%
	DRY DENSITY, LB/CU FT	γ_d	93.1	92.9		
VOID RATIO AFTER CONSOLIDATION		e_c	0.808	0.743		
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	1	1		
FINAL	WATER CONTENT	w_f	28.2 %	27.1 %	%	%
	VOID RATIO	e_f	0.767	0.736		
	SATURATION	S_f	100 %	100 %	%	%
NORMAL STRESS, T/SQ FT		σ	1.00	3.00		
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	0.74	2.19		
ACTUAL TIME TO FAILURE, MIN		t_f	60	190		
RATE OF STRAIN, IN./MIN			.0083	.0026		
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}	-	-		

TYPE OF SPECIMEN Remolded — 3 1/4 IN. SQUARE — 0.50 IN. THICK

CLASSIFICATION Silty Sand (SM)

LL 47 | PL 29 | PI 18 | G_s 2.72

REMARKS Div. No. 92444

Remolded to 90% max. density at 2% above optimum water content.

PROJECT Upper Santa Ana River

AREA

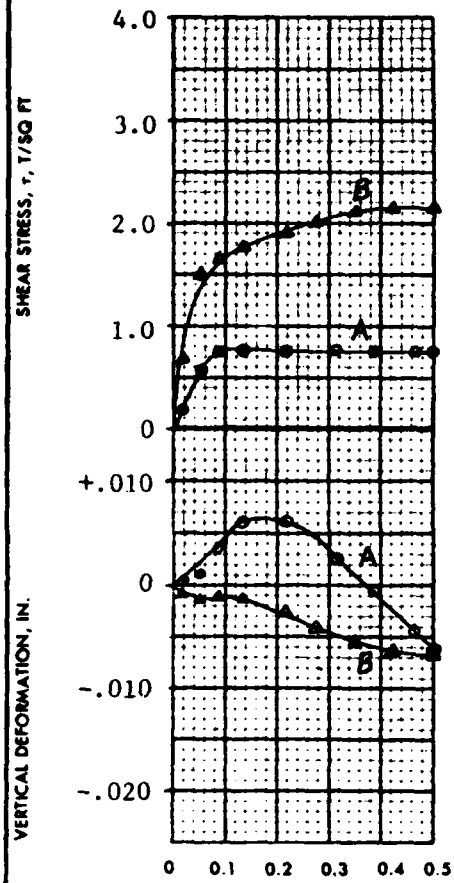
BORING NO. TH-1 | SAMPLE NO. 6

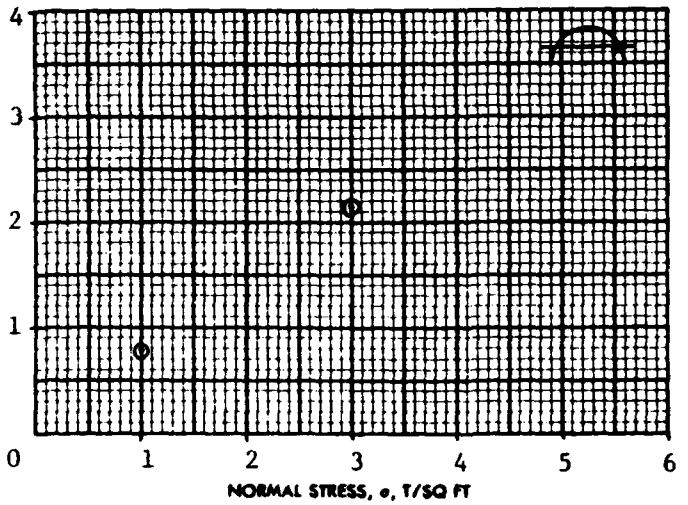
DEPTH EL 18-21 | DATE August 1985

DIRECT SHEAR TEST REPORT

ENG FORM 2092 1 JUN 65 (EM 1110-2-1906) PREVIOUS EDITIONS ARE OBSOLETE (TRANSLUCENT) GPO : 1966 O-214-545

PLATE 7





SHEAR STRENGTH PARAMETERS

ϕ' = _____

TAN ϕ' = _____

c' = _____ T/SQ FT

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

TEST NO.			A	B		
INITIAL	WATER CONTENT	w_o	12.4%	12.8%	%	%
	VOID RATIO	e_o	0.545	0.550		
	SATURATION	S_o	63 %	65%	%	%
	DRY DENSITY, LB/CU FT	γ_d	112.3	111.9		
VOID RATIO AFTER CONSOLIDATION		e_c	0.529	0.518		
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	< 1	< 1		
FINAL	WATER CONTENT	w_f	18.3%	17.9%	%	%
	VOID RATIO	e_f	0.510	0.497		
	SATURATION	S_f	100 %	100 %	%	%
NORMAL STRESS, T/SQ FT		σ	1.00	3.00		
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	0.79	2.15		
ACTUAL TIME TO FAILURE, MIN		t_f	60	210		
RATE OF STRAIN, IN./MIN			.0083	.0024		
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}	-	-		

TYPE OF SPECIMEN Remolded — 3½ IN. SQUARE — 0.50 IN. THICK

CLASSIFICATION Silty Sand (SM)

LL | PL | PI NP | | G_s 2.78

REMARKS Div. No. 92449

Remolded to 90% max. density at 2% above opt. water content.

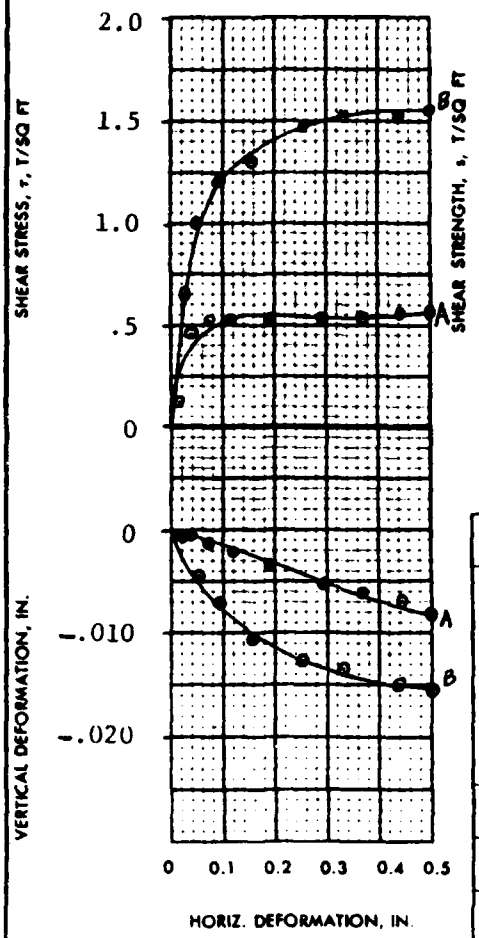
PROJECT Upper Santa Ana River

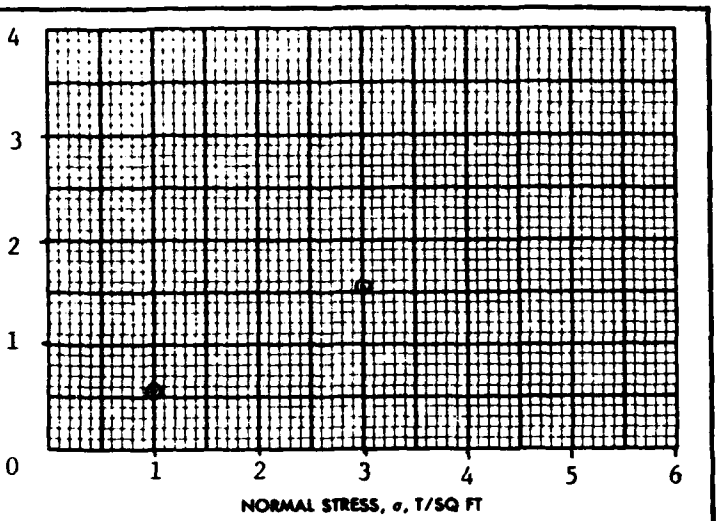
AREA U/S Core Borrow

BORING NO. TH-6 | SAMPLE NO. 4

DEPTH EL 8-11 | DATE Sept. 1985

DIRECT SHEAR TEST REPORT





SHEAR STRENGTH PARAMETERS

ϕ' = _____

TAN ϕ' = _____

c' = _____ T/SQ FT

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

TEST NO.			A	B		
INITIAL	WATER CONTENT	w_o	12.6 %	12.5 %	%	%
	VOID RATIO	e_o	.4955	.4955		
	SATURATION	S_o	70.4 %	69.7 %	%	%
	DRY DENSITY, LB/CU FT	γ_d	115.6	115.6		
VOID RATIO AFTER CONSOLIDATION		e_c	.4671	.4279		
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	<1	<1		
FINAL	WATER CONTENT	w_f	16.0 %	13.8 %	%	%
	VOID RATIO	e_f	.4423	.3813		
	SATURATION	S_f	100 %	100 %	%	%
NORMAL STRESS, T/SQ FT		σ	1	3		
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	.56	1.55		
ACTUAL TIME TO FAILURE, MIN		t_f	215	215		
RATE OF STRAIN, IN./MIN			.0023	0.00		
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}				

TYPE OF SPECIMEN: REMOLDED — 3.25 IN. SQUARE — 0.50 IN. THICK

CLASSIFICATION: Sandy Gravelly Clay (CL)

LL 38 | PL 16 | PI 22 | G_s 2.77

REMARKS: Div. No. 92450
Remolded to 90% max. density at 2% above opt. water content.

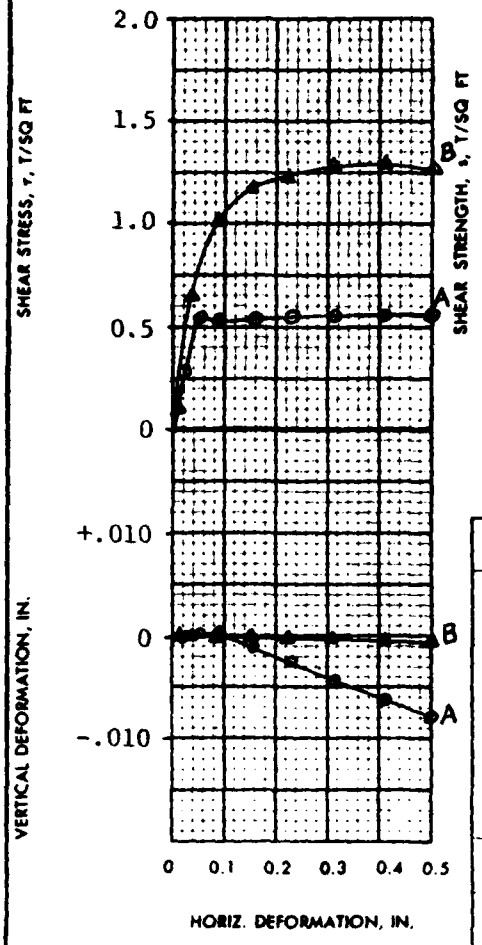
PROJECT: UPPER SANTA ANA RIVER

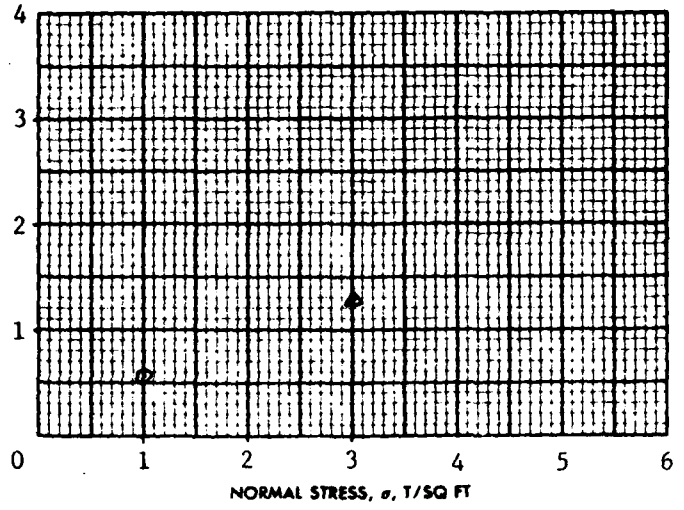
AREA: Upstream Core Borrow Site

BORING NO. TH-7 | SAMPLE NO. 9

DEPTH EL 20-23 | DATE Sept. 1985

DIRECT SHEAR TEST REPORT





TEST NO.			A	B		
INITIAL	WATER CONTENT	w_o	12.8%	13.1%	%	%
	VOID RATIO	e_o	.6063	.6073		
	SATURATION	S_o	59 %	61 %	%	%
	DRY DENSITY, LB/CU FT	γ_d	109.2	109.1		
VOID RATIO AFTER CONSOLIDATION		e_c	.5947	.5823		
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	<1	<1		
FINAL	WATER CONTENT	w_f	20.3 %	20.7%	%	%
	VOID RATIO	e_f	.5693	.5806		
	SATURATION	S_f	100%	100%	%	%
NORMAL STRESS, T/SQ FT		σ	1	3		
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	.560	1.290		
ACTUAL TIME TO FAILURE, MIN		t_f	125	125		
RATE OF STRAIN, IN./MIN			.004	.004		
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}	–	–		

SHEAR STRENGTH PARAMETERS

s' = _____

TAN ϕ' = _____

c' = _____ T/SQ FT

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

TYPE OF SPECIMEN	Remolded		3.25	IN. SQUARE	0.50 IN. THICK
CLASSIFICATION	Sandy Clay (CL)				
LL 35	PL 19	PI 16			G_s 2.81

REMARKS Remolded to 90% max. density at 2% above opt. water content.

Div. No. 92452

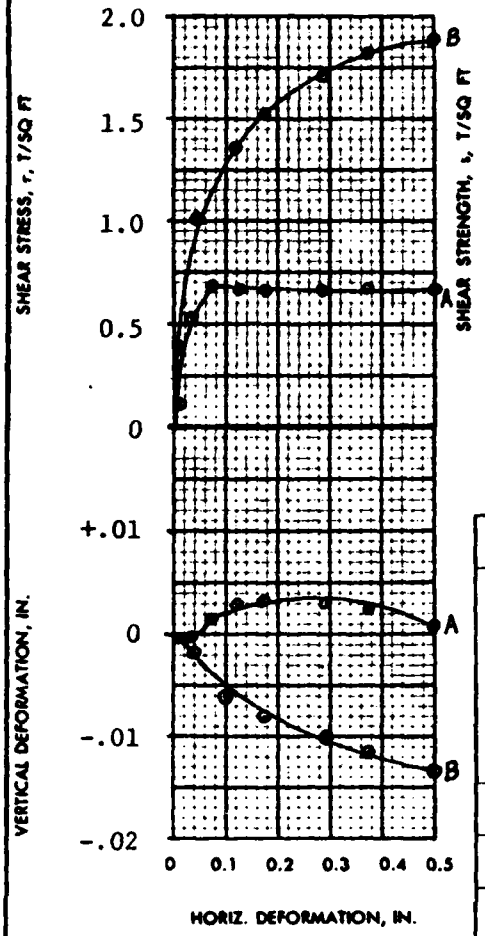
PROJECT UPPER SANTA ANA RIVER

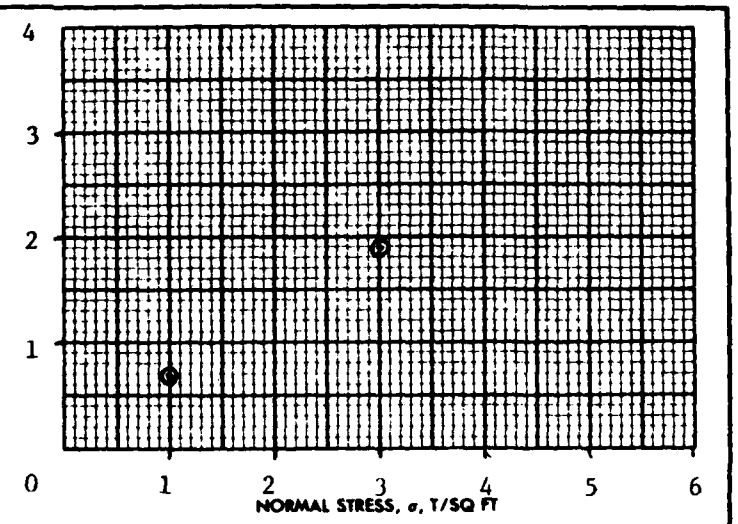
AREA Upstream Core Borrow Site

BORING NO. TH-9 SAMPLE NO. 8

DEPTH EL 18.5 - 20 DATE Sept. 1985

DIRECT SHEAR TEST REPORT





SHEAR STRENGTH PARAMETERS

ϕ' = _____

TAN ϕ' = _____

c' = _____ T/SQ FT

☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

TEST NO.			A	B		
INITIAL	WATER CONTENT	w_o	12.6 %	12.3%	%	%
	VOID RATIO	e_o	.5444	.5414		
	SATURATION	S_o	63.5%	62.5%	%	%
	DRY DENSITY, LB/CU FT	γ_d	111.1	111.3		
VOID RATIO AFTER CONSOLIDATION		e_c	.5321	.5217		
TIME FOR 50 PERCENT CONSOLIDATION, MIN		t_{50}	<1	<1		
FINAL	WATER CONTENT	w_f	19.5%	17.5%	%	%
	VOID RATIO	e_f	.5349	.4810		
	SATURATION	S_f	100%	100%	%	%
NORMAL STRESS, T/SQ FT		σ	1	3		
MAXIMUM SHEAR STRESS, T/SQ FT		τ_{max}	0.69	1.89		
ACTUAL TIME TO FAILURE, MIN		t_f	40	190		
RATE OF STRAIN, IN./MIN			0.00	0.00		
ULTIMATE SHEAR STRESS, T/SQ FT		τ_{ult}				

TYPE OF SPECIMEN REMOLDED — 3.25 IN. SQUARE — 0.50 IN. THICK

CLASSIFICATION Sandy Clay (CL)

LL 41 | PL 21 | PI 20 | G_s 2.75

REMARKS Div. No. 92453
Remolded to 90% max. density at 2% above opt. water content.

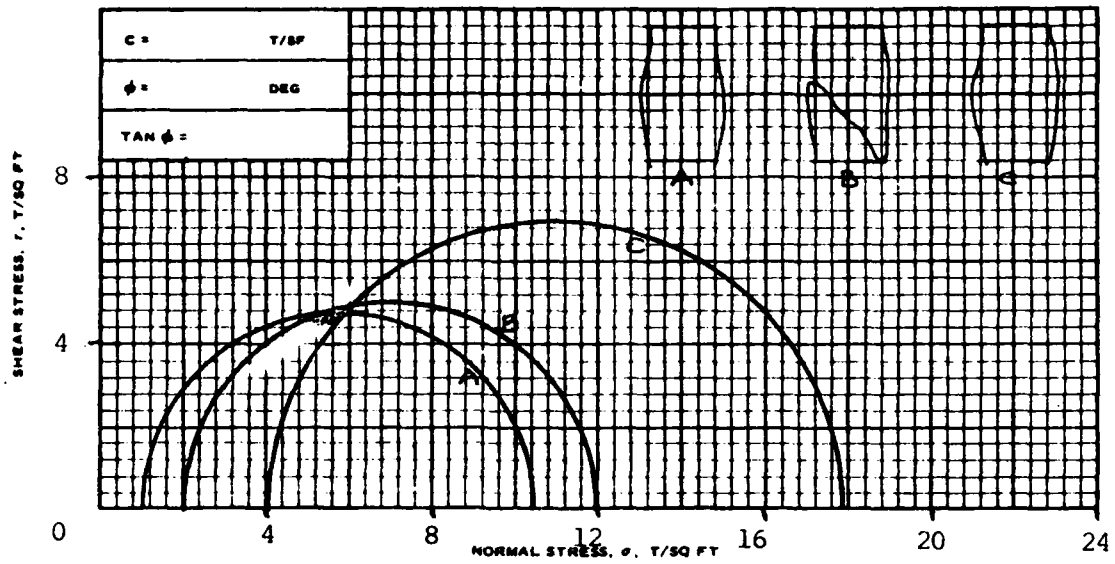
PROJECT UPPER SANTA ANA RIVER

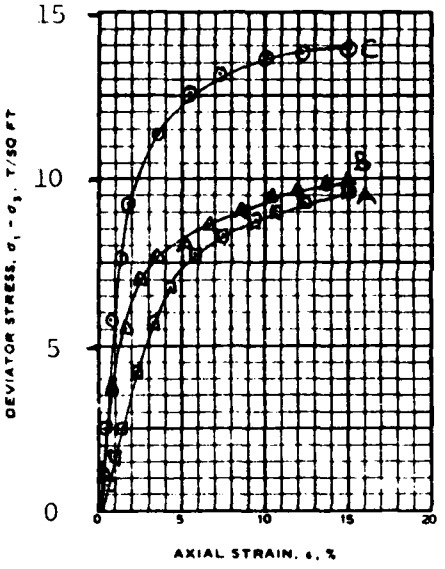
AREA Upstream Core Borrow Site

BORING NO. TH-10 | SAMPLE NO. 16

DEPTH EL 30-34 | DATE Sept. 1985

DIRECT SHEAR TEST REPORT





SPECIMEN NO.			A	B	C
INITIAL	WATER CONTENT, %	w_o	11.9	12.2	12.2
	DRY DENSITY LB/ CU FT	γ_{d_o}	116.9	116.5	116.4
	SATURATION, %	S_o	67	68	68
	VOID RATIO	e_o	0.500	0.505	0.507
BEFORE SHEAR	WATER CONTENT, %	w_c	16.6	17.4	17.2
	DRY DENSITY LB/CU FT	γ_{d_c}	119.5	117.8	118.3
	SATURATION, %	S_c	100	100	100
	VOID RATIO	e_c	0.467	0.489	0.482
	FINAL BACK PRESSURE, T/SQ FT	u_o	7.20	7.20	7.20
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	1.00	2.00	4.00
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	9.59	9.99	13.99
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	285	297	222
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$	–	–	–
INITIAL DIAMETER, IN.		D_o	2.8	2.8	2.8
INITIAL HEIGHT, IN		H_o	6.45	6.45	6.45

CONTROLLED- Strain TEST

DESCRIPTION OF SPECIMENS Sandy Silty Gravel (GM)

29% + 4-1/2" gravel

LL	PL	PI	G_s 2.81	TYPE OF SPECIMEN Remolded	TYPE OF TEST R

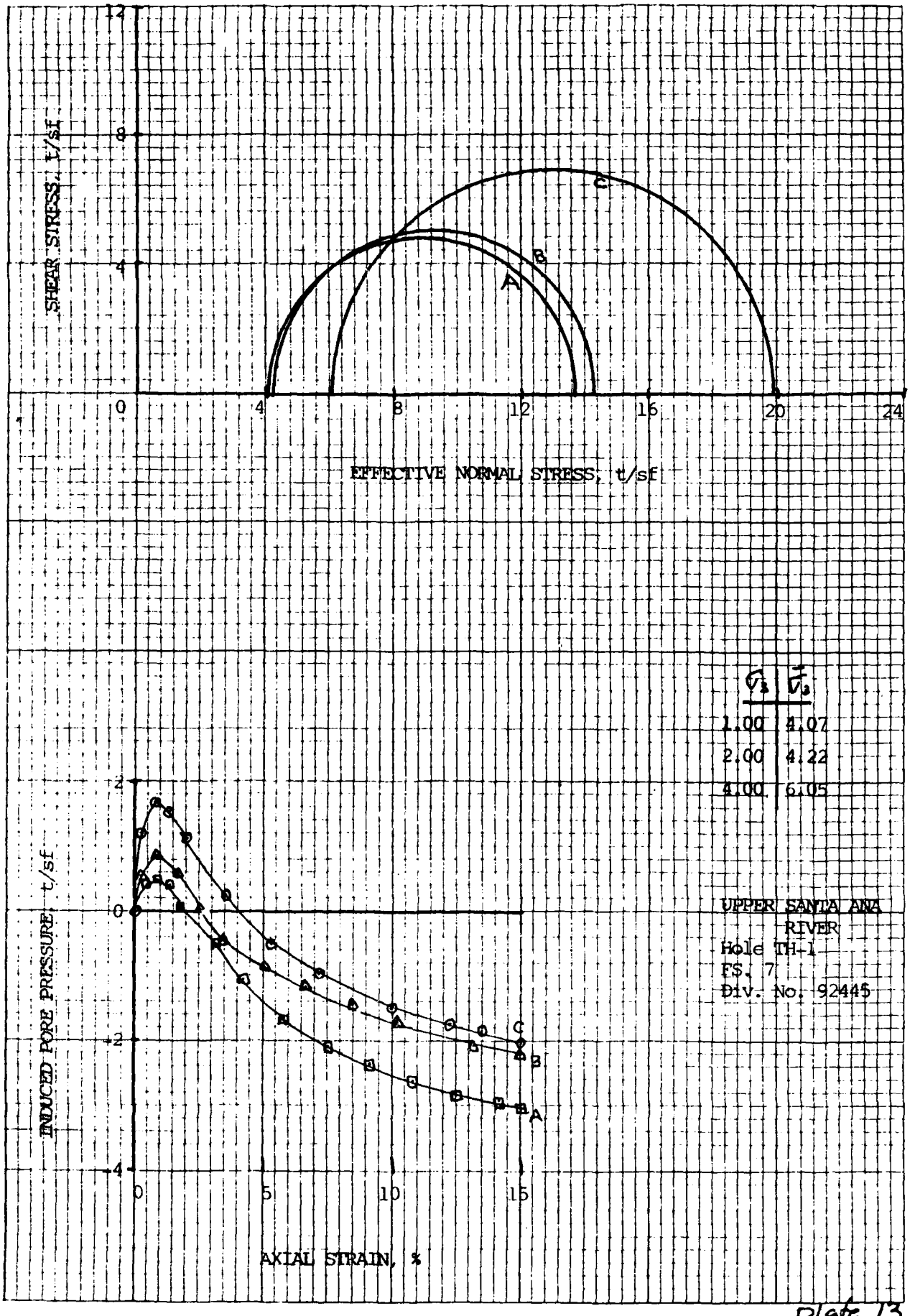
REMARKS: Div. No. 92445
Remolded to 90% max. density at
2% above optimum water content.

PROJECT UPPER SANTA ANA RIVER
U/S Core Borrow
BORING NO TH-1 SAMPLE NO. 7
DEPTH/ELEV 27-30
LABORATORY SPDL DATE October 1985

TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT (EM 1110-2-1906)

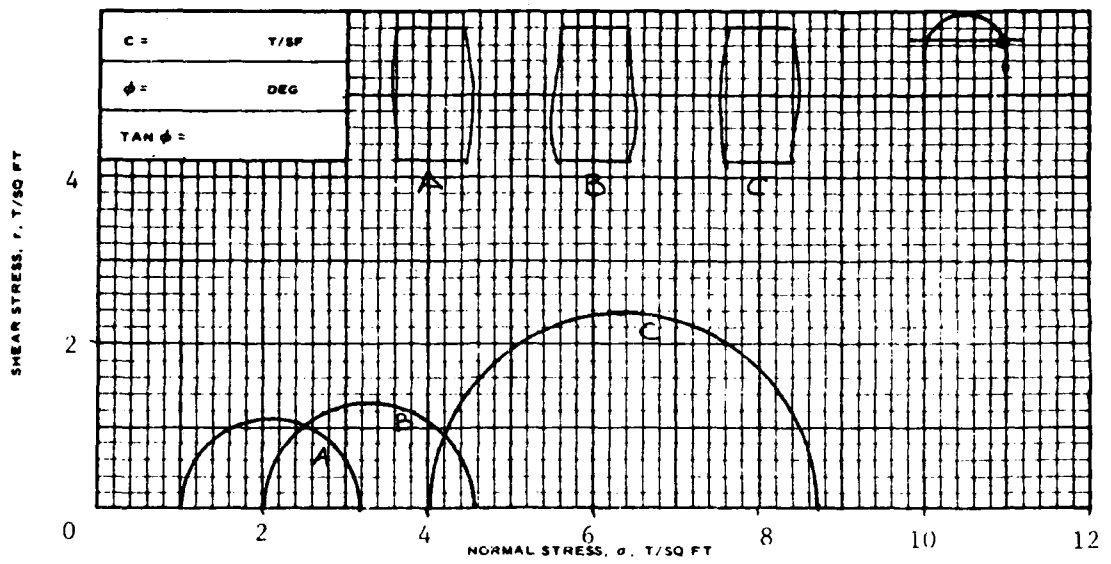
plate 12

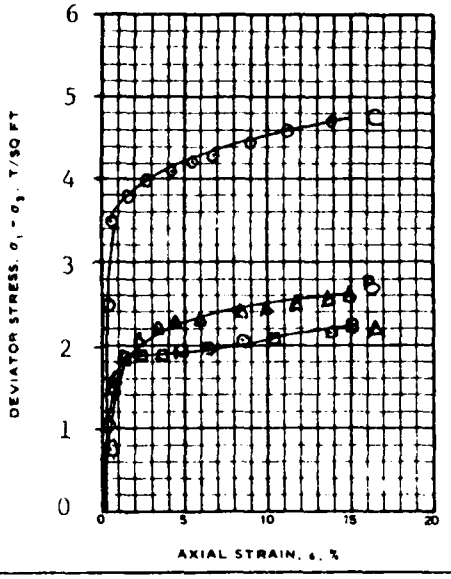


σ_3	$\bar{\sigma}_3$
1.00	4.07
2.00	4.22
4.00	6.05

UPPER SANTA ANA RIVER
Hole TH-1
FS. 7
Div. No. 92445

plate 13





SPECIMEN NO.			A	B	C
INITIAL	WATER CONTENT, %	w_o	13.4	13.1	13.3
	DRY DENSITY LB/CU FT	γ_{d_o}	114.5	114.9	114.7
	SATURATION, %	S_o	73	73	73
	VOID RATIO	e_o	0.505	0.499	0.502
BEFORE SHEAR	WATER CONTENT, %	w_c	18.1	17.3	16.9
	DRY DENSITY LB/CU FT	γ_{d_c}	114.9	116.6	117.5
	SATURATION, %	S_c	100	100	100
	VOID RATIO	e_c	0.499	0.477	0.460
	FINAL BACK PRESSURE, T/SQ FT	u_o	7.20	7.20	7.20
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	1.00	2.00	4.00
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	2.26	2.61	4.73
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	243	253	220
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$	-	-	-
INITIAL DIAMETER, IN.		D_o	2.8	2.8	2.8
INITIAL HEIGHT, IN.		H_o	6.45	6.45	6.45

CONTROLLED- Strain TEST

DESCRIPTION OF SPECIMENS Gravelly Silty Clay (CL) - Specimen contains 14% +4 -1/2" gravel.

LL	PL	PI	G_s
37	23	14	2.76

TYPE OF SPECIMEN Remolded TYPE OF TEST R

REMARKS: Div. No. 92448
Remolded to 90% max. density at 2% above optimum water content.

PROJECT UPPER SANTA ANA RIVER
U/S Core Borrow

BORING NO. TH-5 SAMPLE NO 19

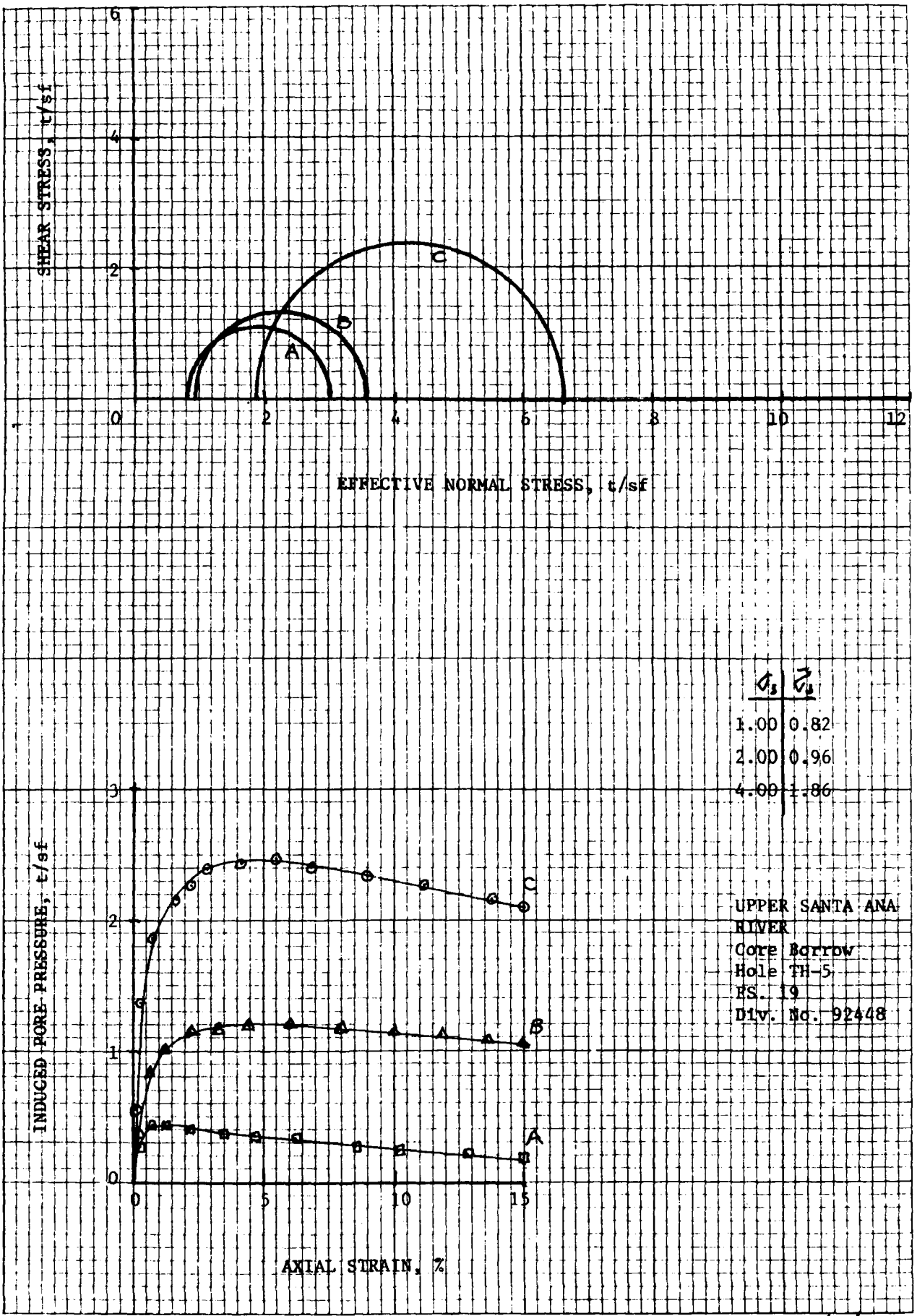
DEPTH ELEV 55.5 - 57.0

LABORATORY SPDL DATE September 1985

TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT (EM 1110-2-1906)

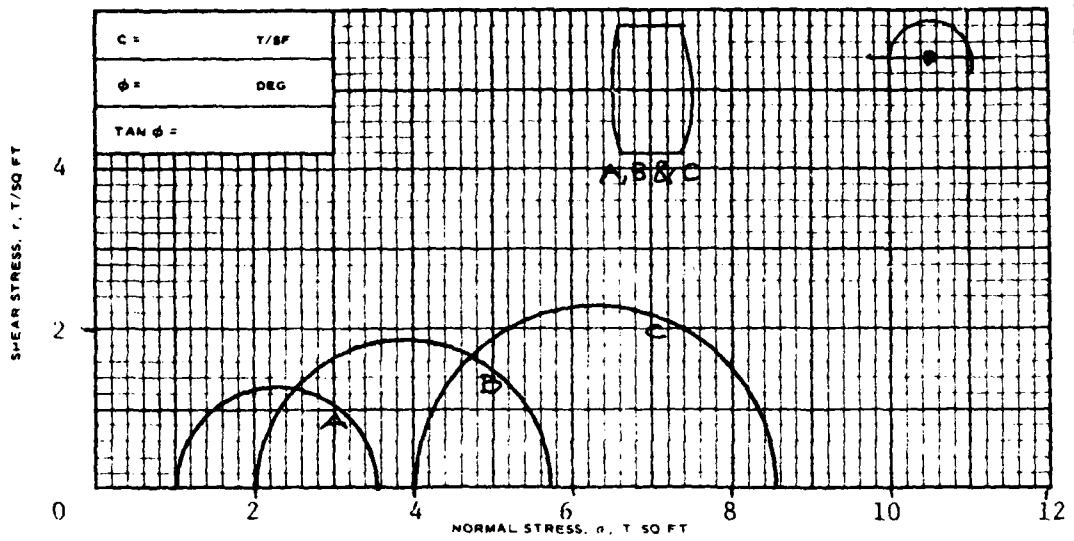
plate 14

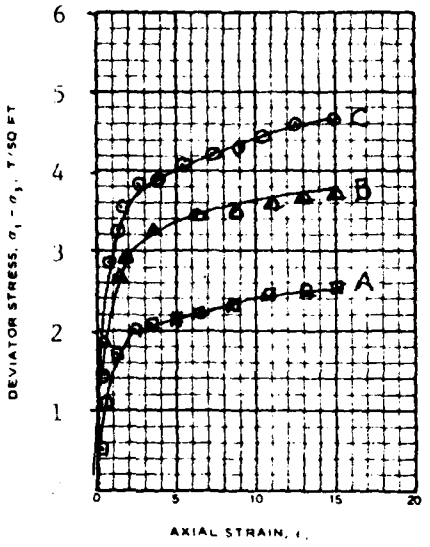


σ_3	τ_d
1.00	0.82
2.00	0.96
4.00	1.86

UPPER SANTA ANA RIVER
Core Borrow
Hole TH-5
ES. 19
Div. No. 92448

Plate 15





SPECIMEN NO			A	B	C
INITIAL	WATER CONTENT, %	w_o	10.9	10.8	10.8
	DRY DENSITY LB/CU FT	γ_{d_o}	118.6	118.7	118.6
	SATURATION, %	S_o	65	65	65
	VOID RATIO	e_o	0.463	0.462	0.463
BEFORE SHEAR	WATER CONTENT, %	w_c	16.3	15.9	15.8
	DRY DENSITY LB/CU FT	γ_{d_c}	119.5	120.2	120.6
	SATURATION, %	S_c	100	100	100
	VOID RATIO	e_c	0.452	0.443	0.439
	FINAL BACK PRESSURE, T/SQ FT	u_b	7.20	7.20	7.20
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	1.00	2.00	4.00
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	2.55	3.75	4.61
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	272	207	261
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$	–	–	–
INITIAL DIAMETER, IN		D_o	2.8	2.8	2.8
INITIAL HEIGHT, IN		H_o	6.45	6.45	6.45

CONTROLLED- Strain TEST

DESCRIPTION OF SPECIMENS Sandy Clay (CL) — 12% +4-1/2"

LL 33 | PL 21 | PI 12 | G_s 2.78

TYPE OF SPECIMEN Remolded | TYPE OF TEST $\bar{R}$

REMARKS: Div. No. 92451
Remolded to 90% max. density at optimum water content.

PROJECT UPPER SANTA ANA RIVER
U/S Core Borrow

BORING NO. TH-7 | SAMPLE NO 12

DEPTH ELEV 27-28

LABORATORY SPDL | DATE September 1985

TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089
REV JUNE 1970

plate 16

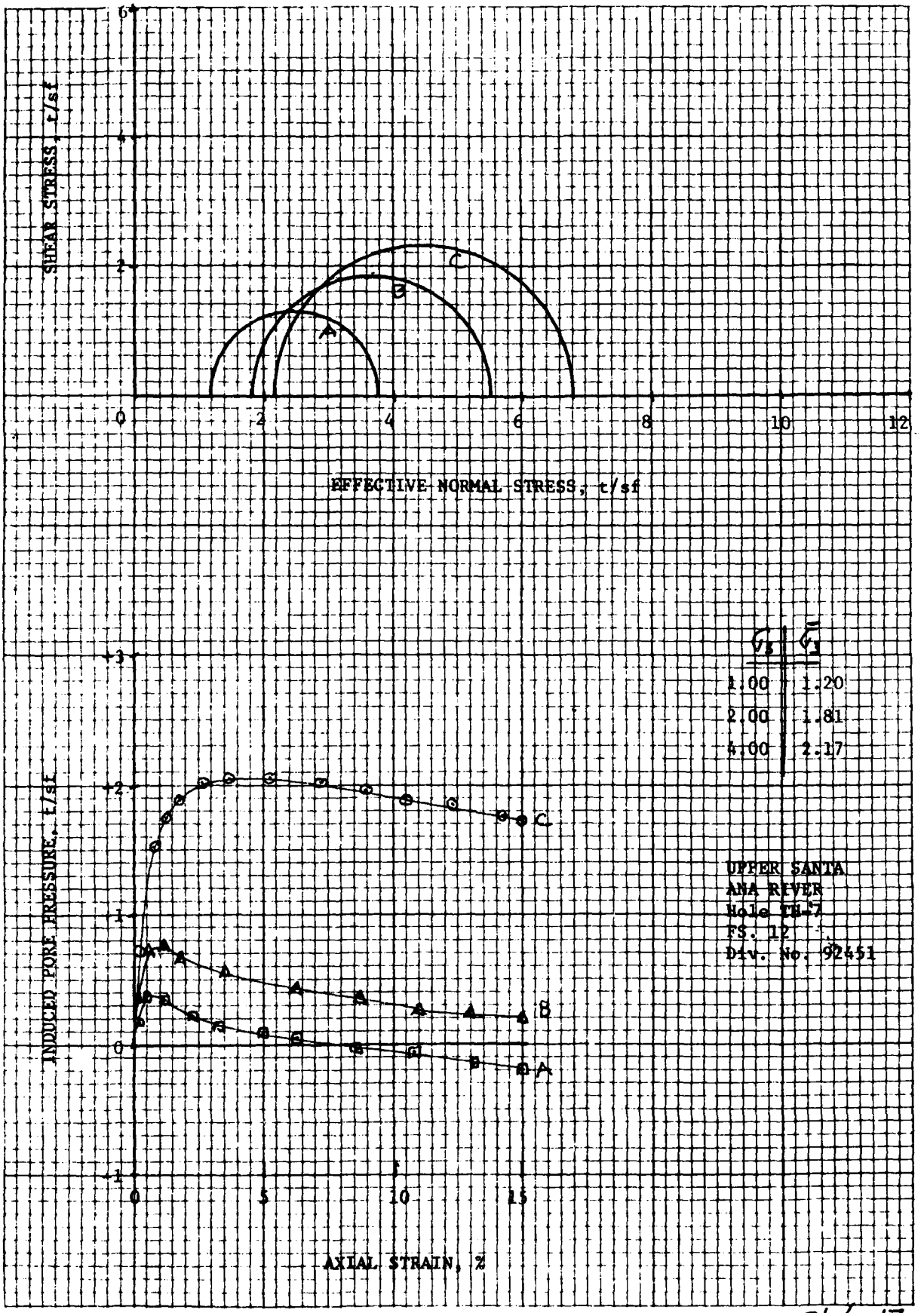
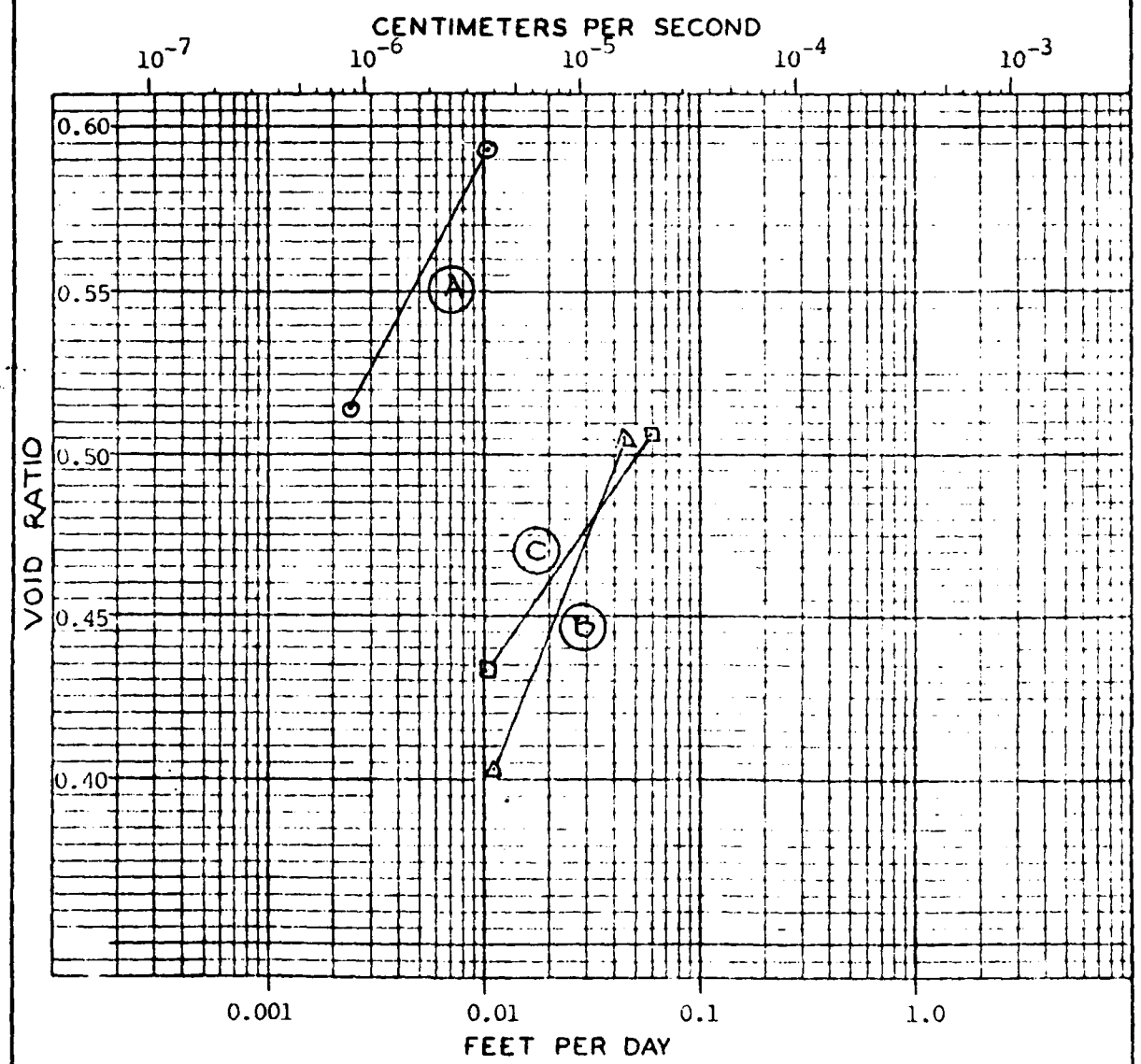

SHEAR STRESS, t/sf
EFFECTIVE NORMAL STRESS, t/sf
INDUCED PORE PRESSURE, t/sf
AXIAL STRAIN, %
A
B
C
σ3 | σ1
1.00 | 1.20
2.00 | 1.81
4.00 | 2.17
UPPER SANTA ANA RIVER
Hole TR-7
FS. 12
Div. No. 92451

plate 17

CORPS OF ENGINEERS — U. S. ARMY

SOUTH PACIFIC DIVISION LABORATORY

PERMEABILITY



REMARKS:

CURVE	SPECIMEN DIAM. IN.	HT. IN.	MAX. PARTICLE	CONDITION
A	4.0	1.0	1/2-in.	Remolded
B	4.0	1.0	1/2-in.	Remolded
C	4.0	1.0	1/2-in.	Remolded

DISTRICT: Los Angeles

PROJECT: Upper Santa Ana River

CURVE	DIV. NO.	HOLE NO.	F.S. NO.	DEPTH FROM	DEPTH TO
A	92444	TH-1	6	18	21
B	92445	TH-1	7	27	30
C	92448	TH-5	19	55.5	57

TESTED	COMPUTED	DRAWN	CHECKED
DW	MC	MC	

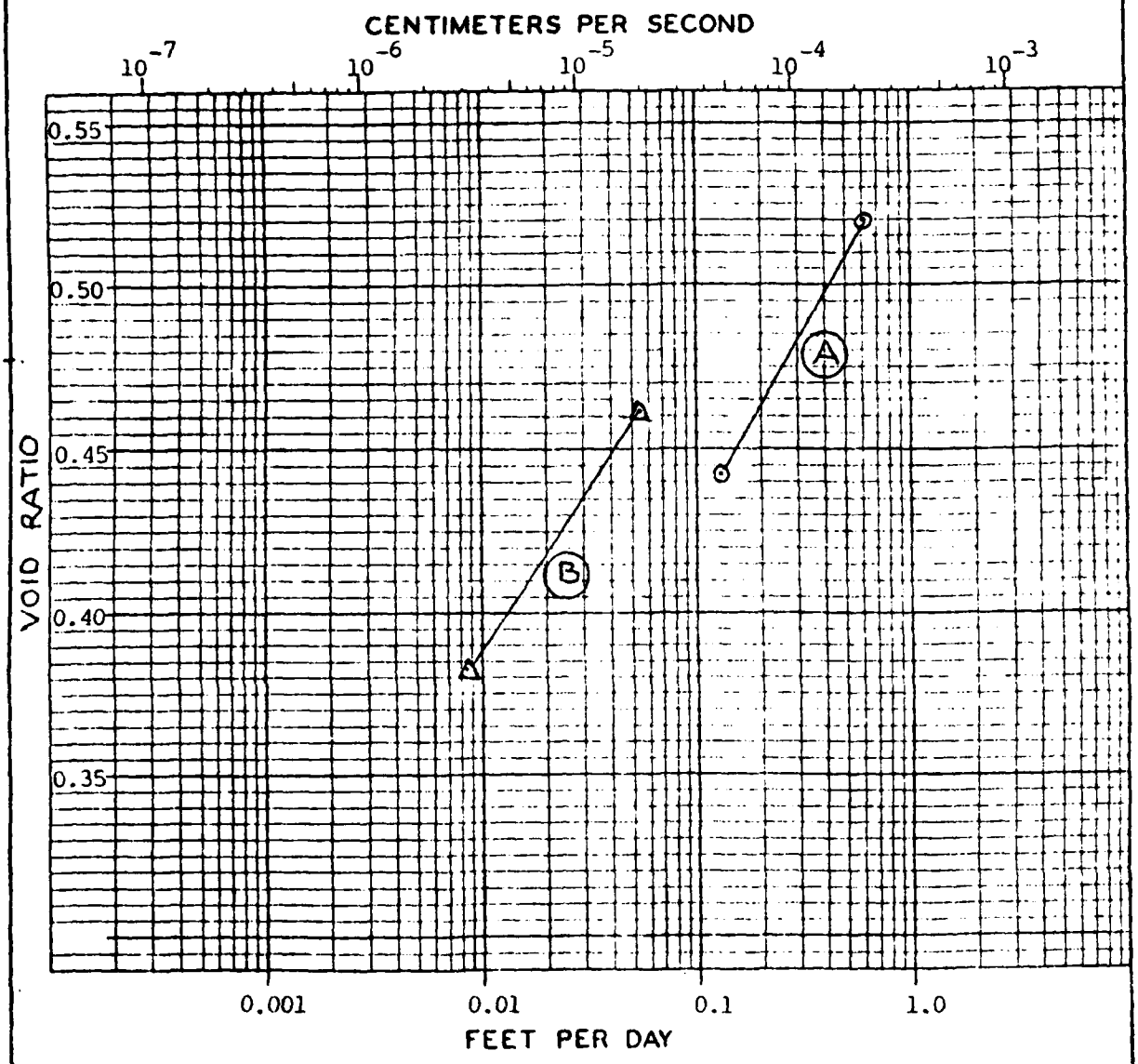
SPD FORM 6 OCT 55 67 (CIVIL) REPLACES PSD 67 (CIVIL), 20 JULY 49, WHICH MAY BE USED.

PLATE 18

CORPS OF ENGINEERS — U. S. ARMY

SOUTH PACIFIC DIVISION LABORATORY

PERMEABILITY



REMARKS:

CURVE	SPECIMEN DIAM. IN.	HT. IN.	MAX. PARTICLE	CONDITION
A	4.0	1.0	No. 10	Remolded
B	4.0	2.0	1/2-in.	Remolded

DISTRICT: Los Angeles

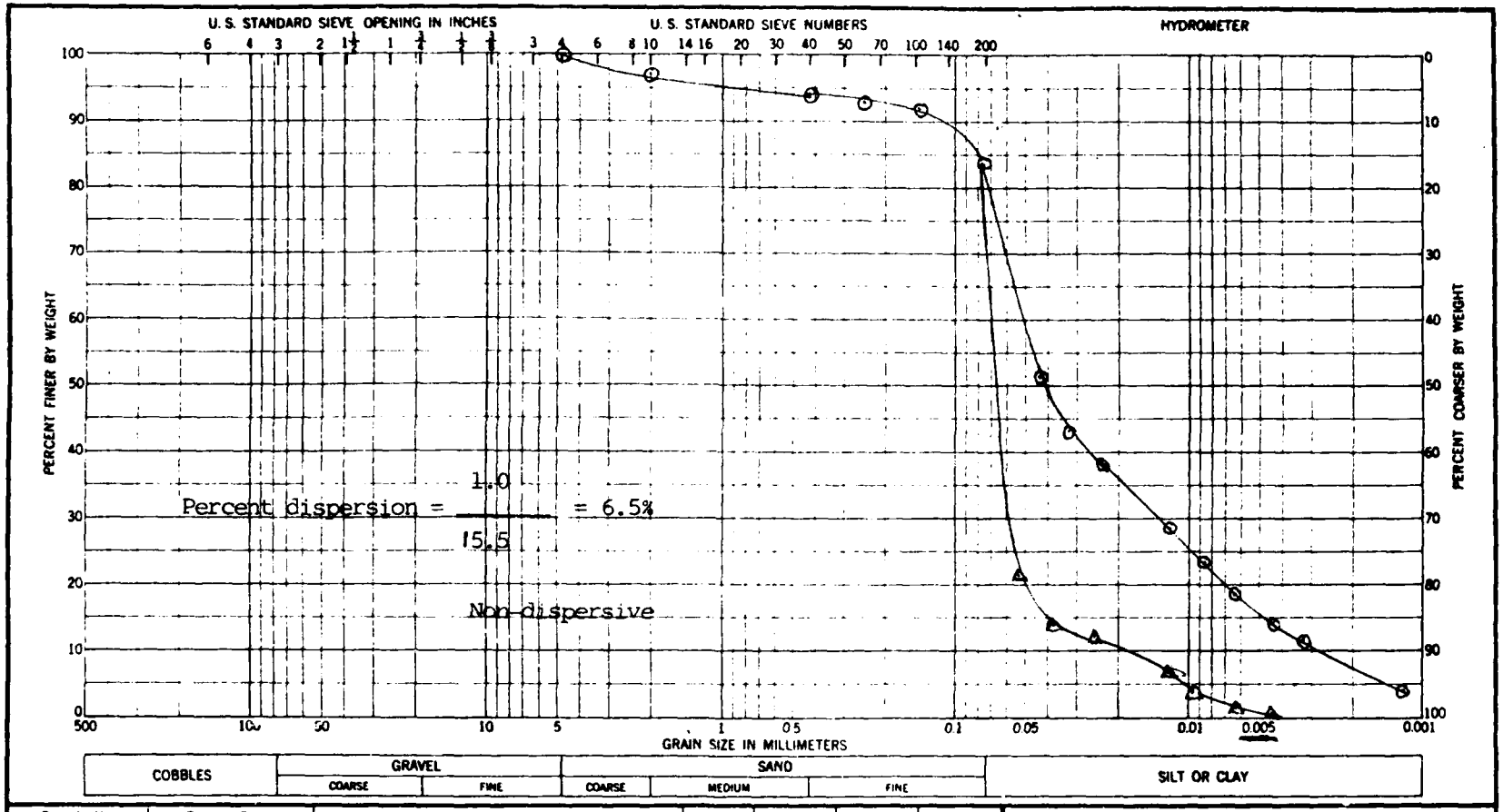
PROJECT: Upper Santa Ana River

CURVE	DIV. NO.	HOLE NO.	F.S. NO.	DEPTH FROM	DEPTH TO
A	92449	TH-6	4	8	11
B	92451	TH-7	12	27	28

TESTED	COMPUTED	DRAWN	CHECKED
DW	MC	MC	

SPD FORM 6 OCT 55 67 (CIVIL) REPLACES PSD 67 (CIVIL), 20 JULY 49, WHICH MAY BE USED.

PLATE 19



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
92446	30-33	Sandy Gravelly Clay		35	20	15

Project Upper Santa Ana River

Area U/S Core Borrow

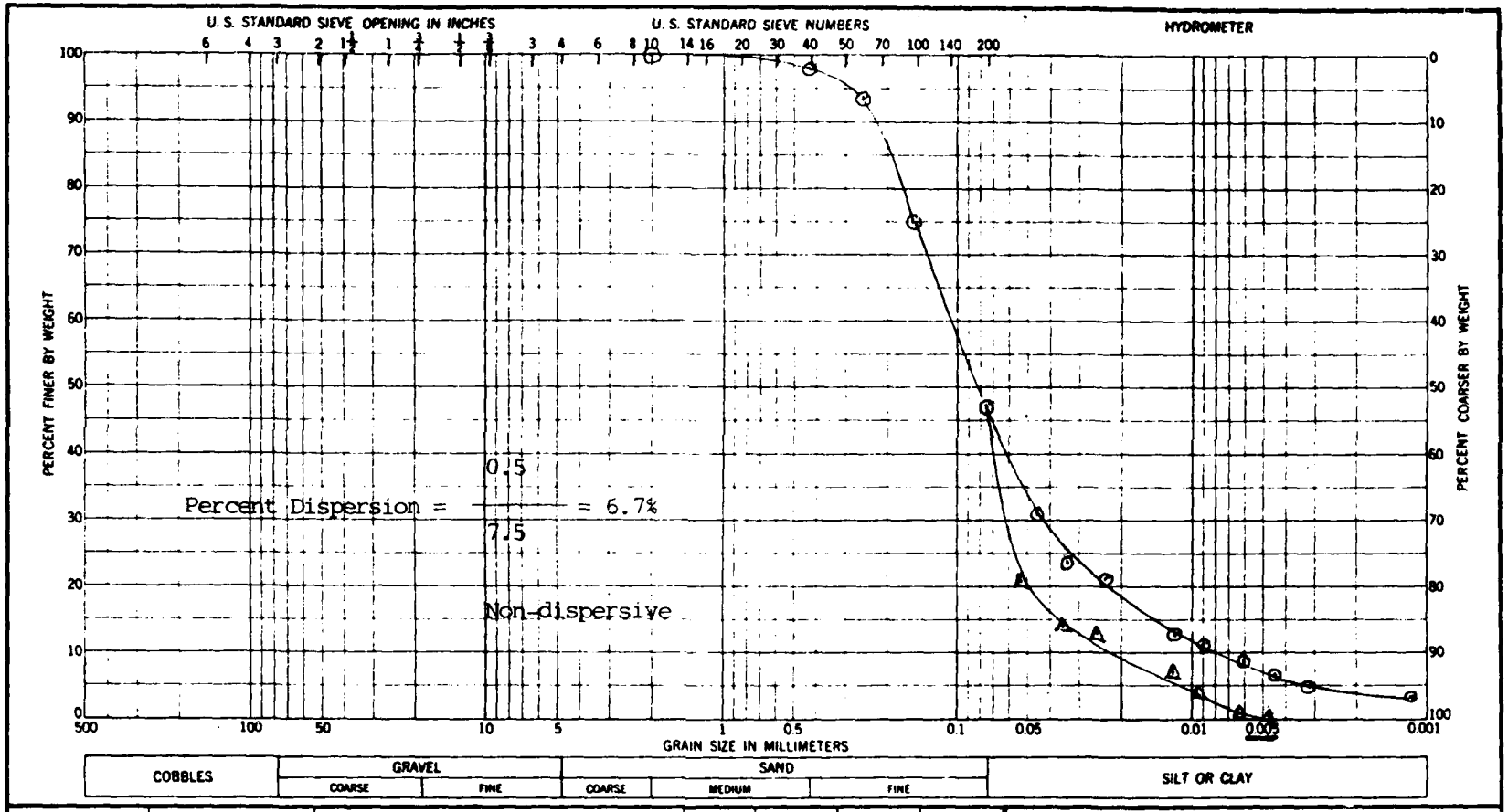
Boring No. TH-5, FS 11

Date November 1985

GRADATION CURVES

ENG FORM 1 MAY 63 2087

Plate 20



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
92447	47.5-50.0	Clayey Sand(SC-SM)		27	20	7

Project: Upper Santa Ana River

Area: U/S Core Borrow

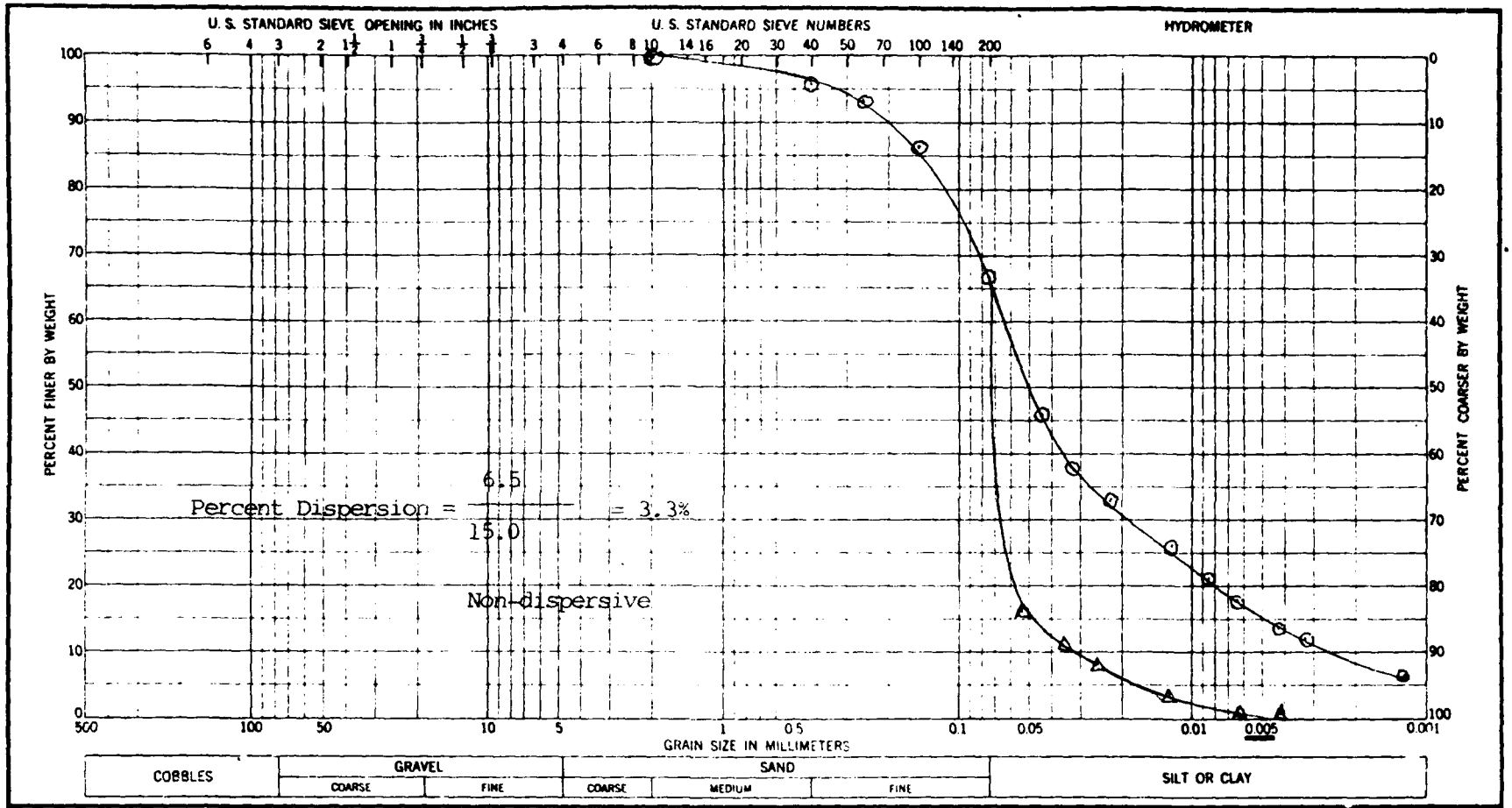
Boring No.: TH-5, FS 16

Date: November 1985

GRADATION CURVES

ENG FORM 1 MAY 63 2087

Plate 21



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
92452	18.5-20.0	Sandy Clay (CL)		35	19	16

Project Upper Santa Ana River

Area U/S Core Borrow

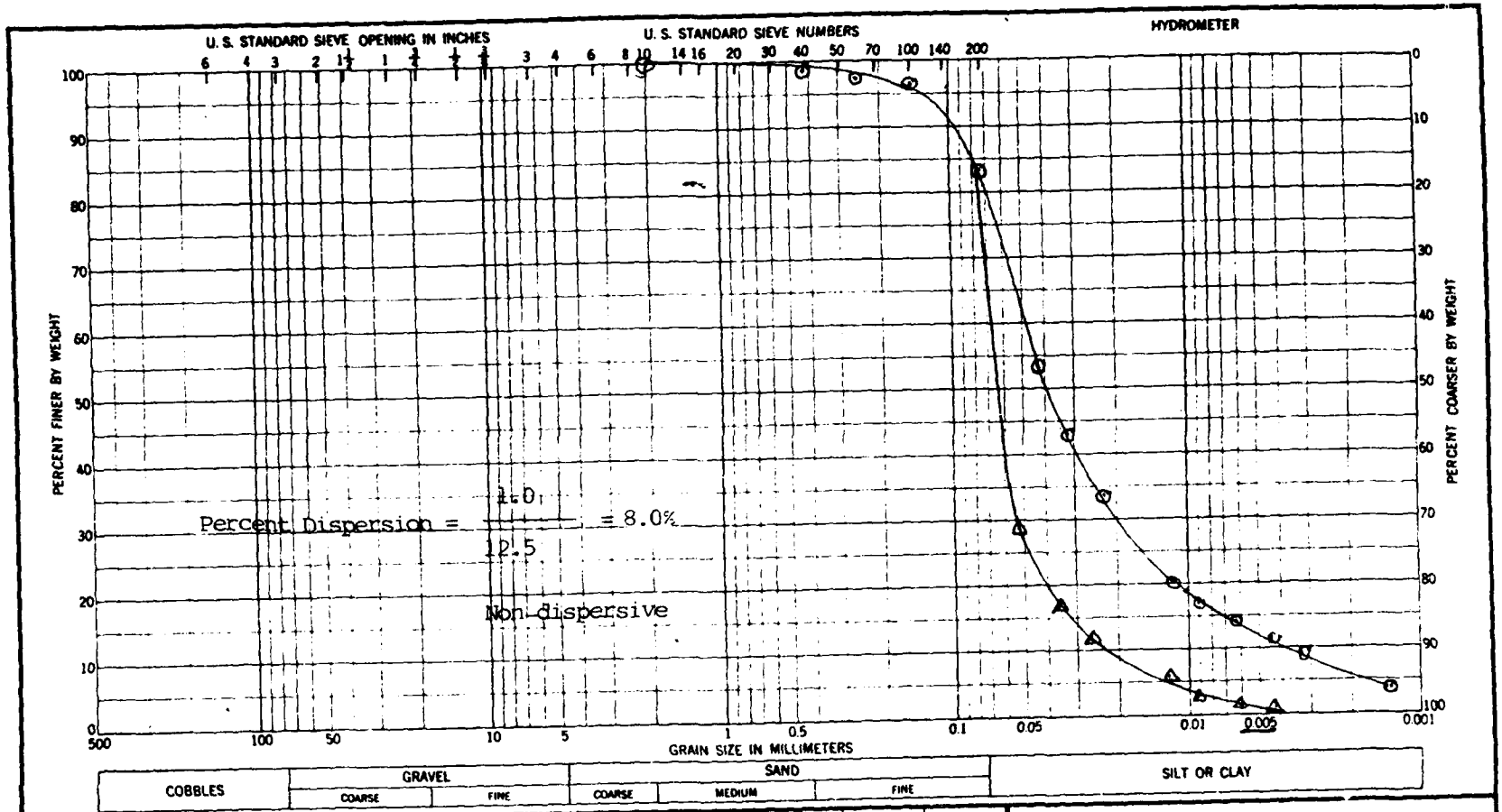
Boring No. TH-9, FS 8

Date November 1985

GRADATION CURVES

ENG FORM 1 MAY 63 2087

Plate 22



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
92450	30-34	Sandy Clay (CL)		41	21	20

Project: Upper Santa Ana River

Area: U/S Core Borrow

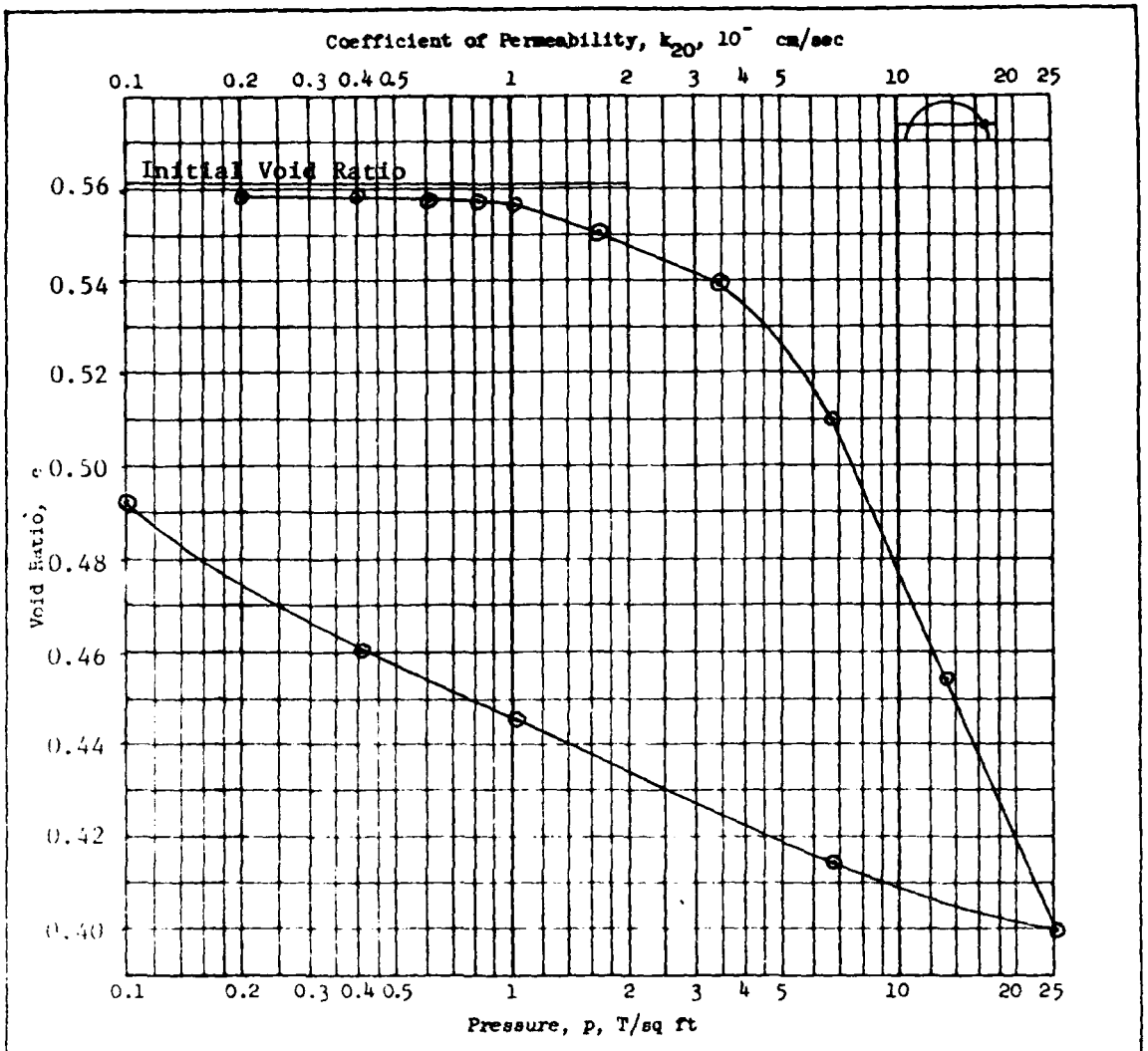
Boring No.: TH-10, FS 16

Date: November 1985

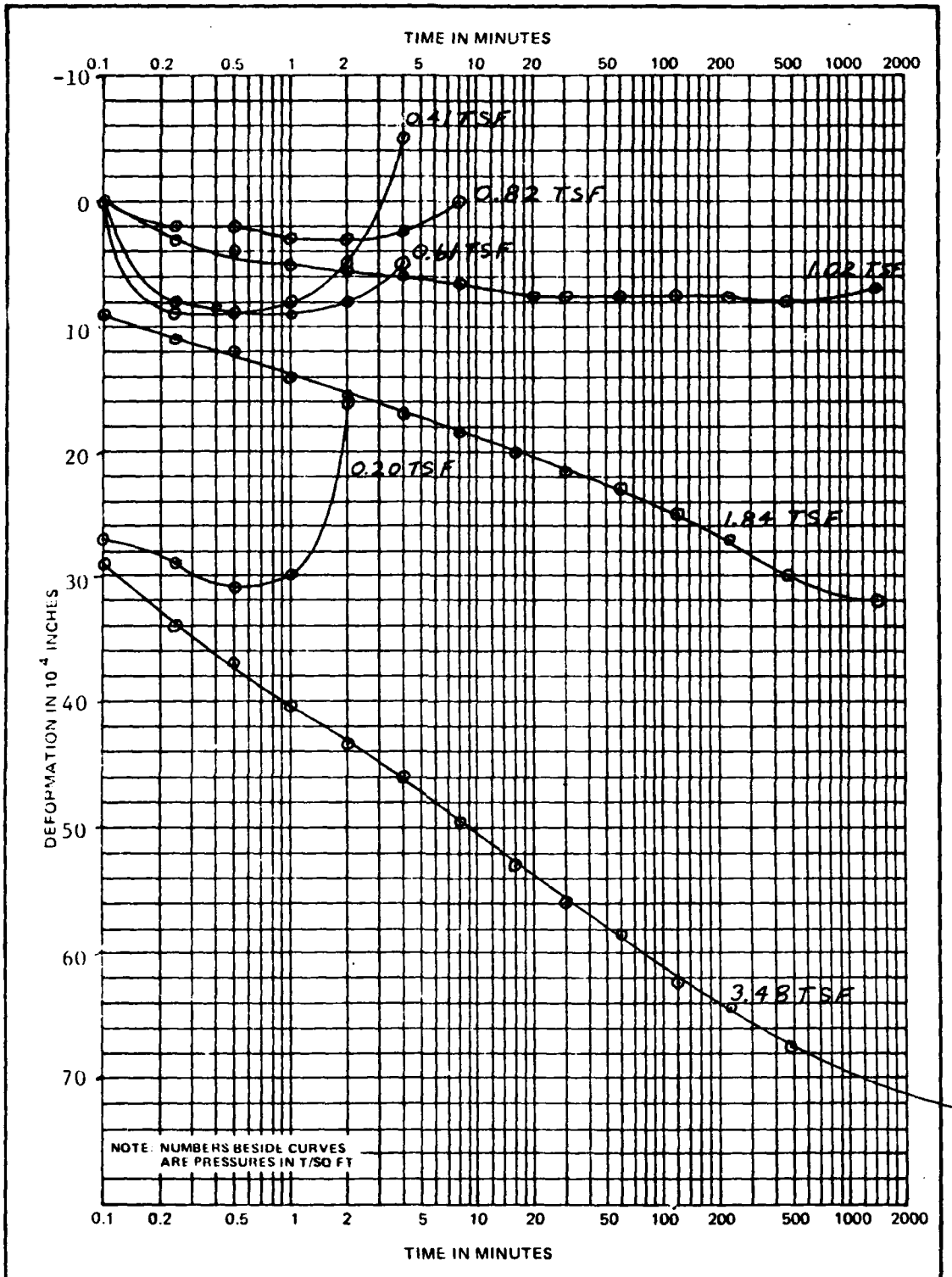
GRADATION CURVES

ENG FORM 1 MAY 63 2087

Plate 23



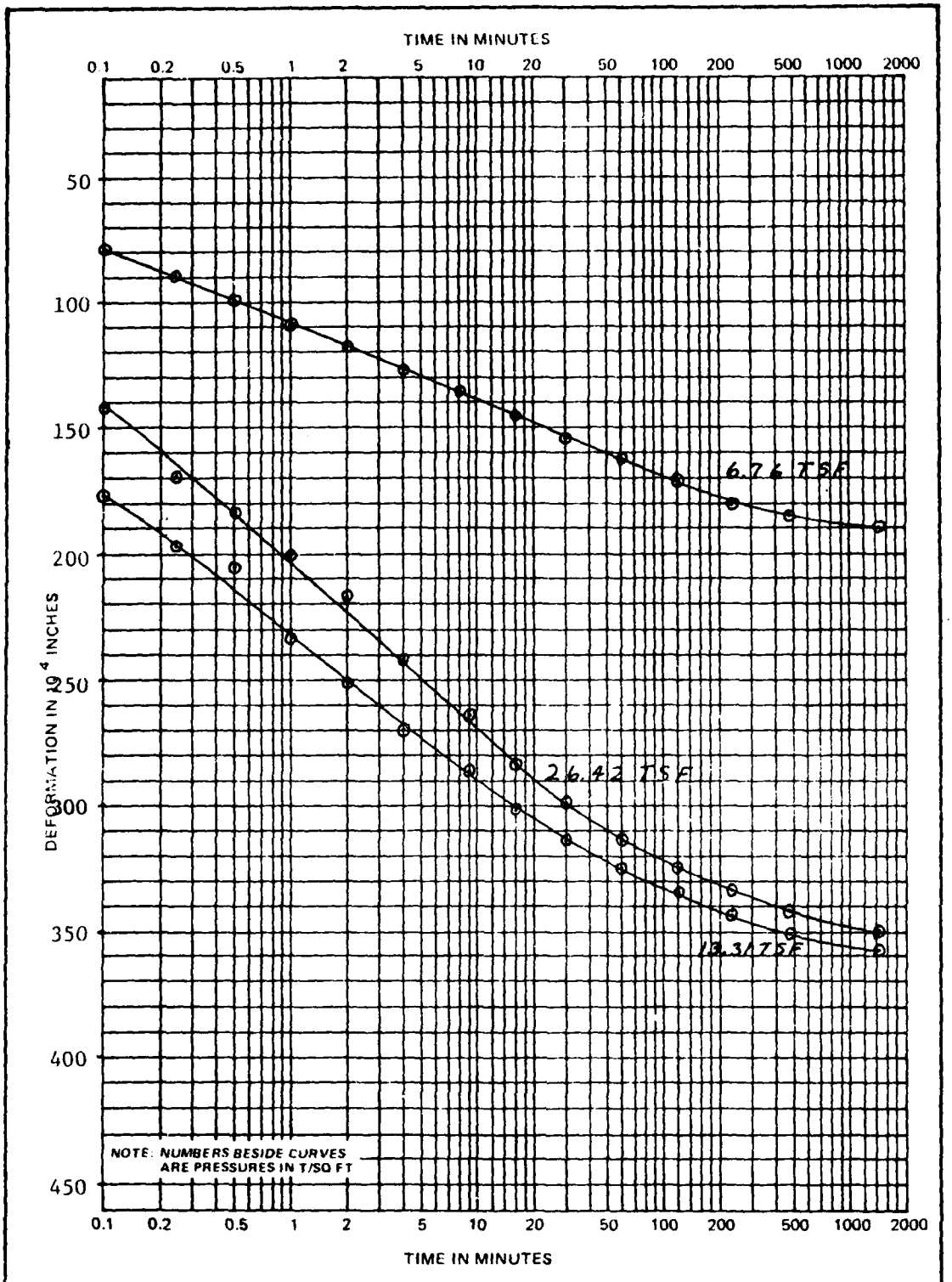
Type of Specimen Remolded		Before Test		After Test	
Diam 4.4 in.	Ht 1.0 in.	Water Content, w_o	12.5 %	w_f	19.2 %
Overburden Pressure, p_o T/sq ft		Void Ratio, e_o	0.561	e_f	0.564
Preconsol. Pressure, p_c T/sq ft		Saturation, S_o	62 %	S_f	95 %
Compression Index, C_c		Dry Density, γ_d	111.5 lb/ft^3		
Classification Sandy Gravelly Clay		k_{20} at e_o =	x 10^{-} cm/sec		
LL 35	G_s 2.79	Project Upper Santa Ana River			
PL 20	D_{10}				
Remarks Div. No. 92446		Area U/S Core Borrow			
Sample remolded to 90% max.		Boring No. TH-5		Sample No. 11	
density at 2% above opt.		Depth El 30-33		Date September 1985	
moisture.		CONSOLIDATION TEST REPORT			



PROJECT Upper Santa Ana River			
AREA U/S Core Borrow			
BORING NO. TH-5	SAMPLE NO. 11	DEPTH EL 30-33	DATE Sept. 1985

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST–TIME CURVES** (TRANSLUCENT)

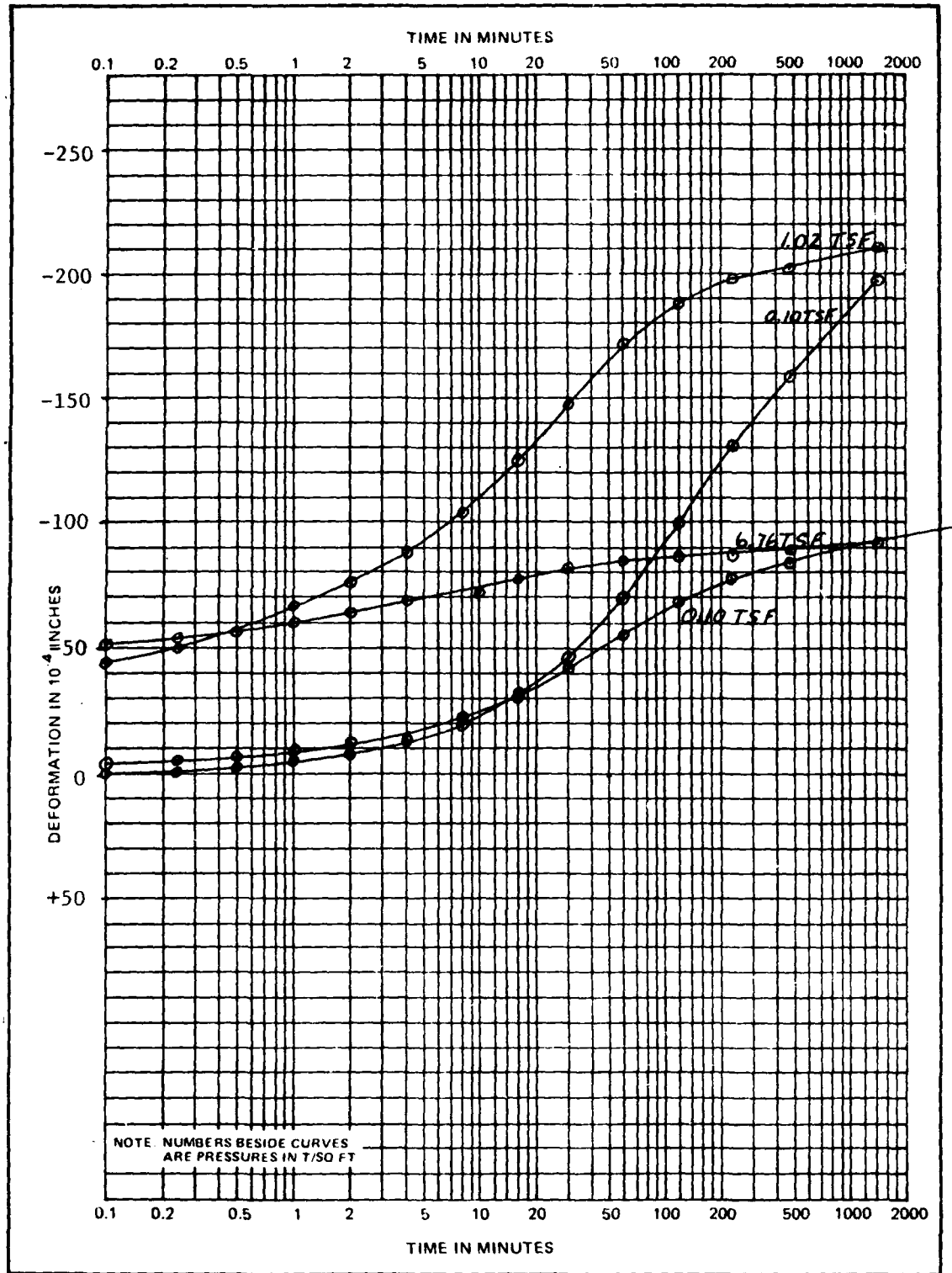
plate 25



PROJECT Upper Santa Ana River			
AREA U/S Core Borrow			
BORING NO. TH-5	SAMPLE NO 11	DEPTH EL 30-33	DATE Sept. 1985

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST–TIME CURVES** (TRANSLUCENT)

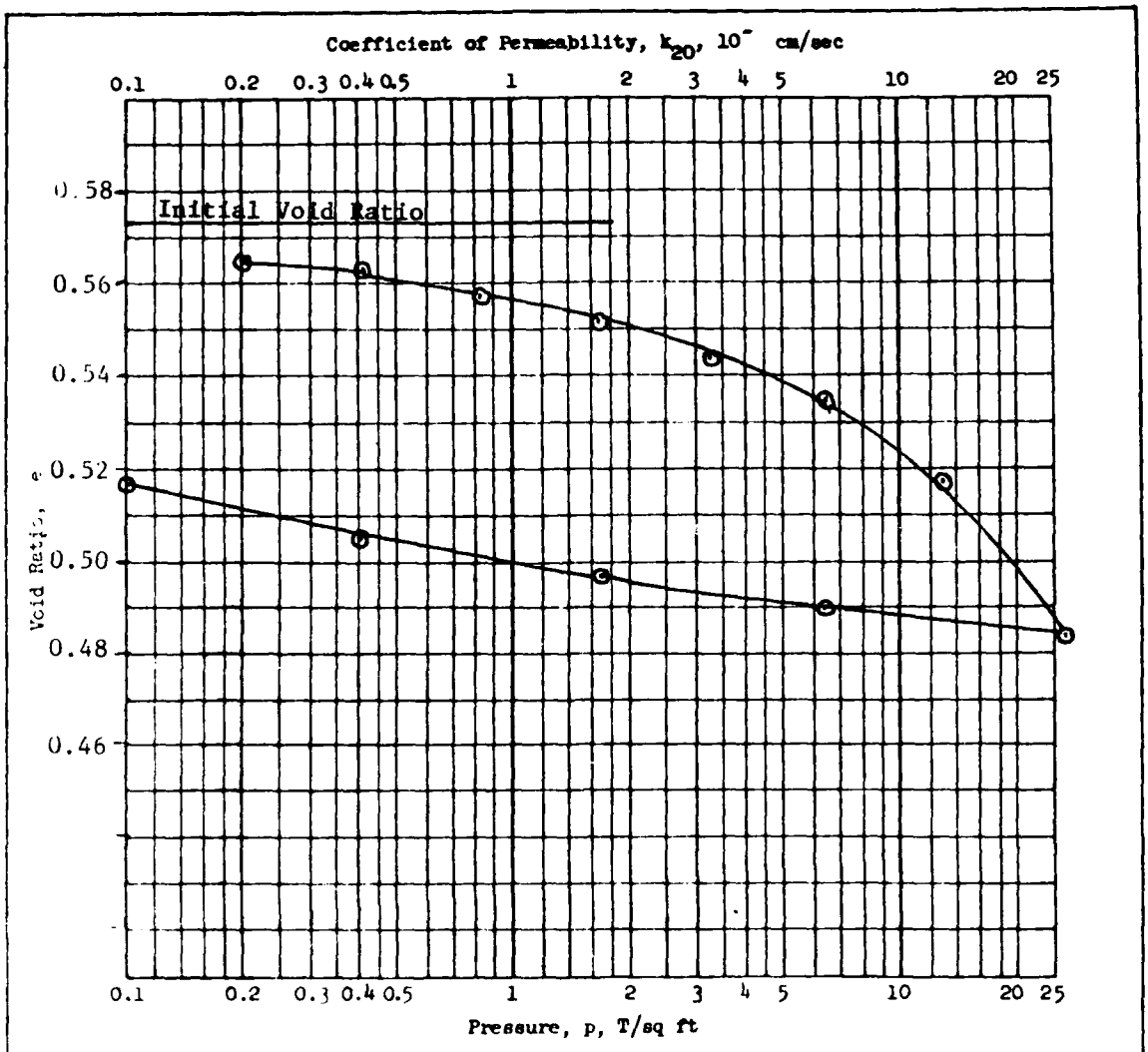
plate 26



PROJECT Upper Santa Ana River			
AREA U/S Core Borrow			
BORING NO. TH-5	SAMPLE NO 11	DEPTH EL 30-33	DATE Sept. 1985

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST–TIME CURVES** (TRANSLUCENT)

plate 27



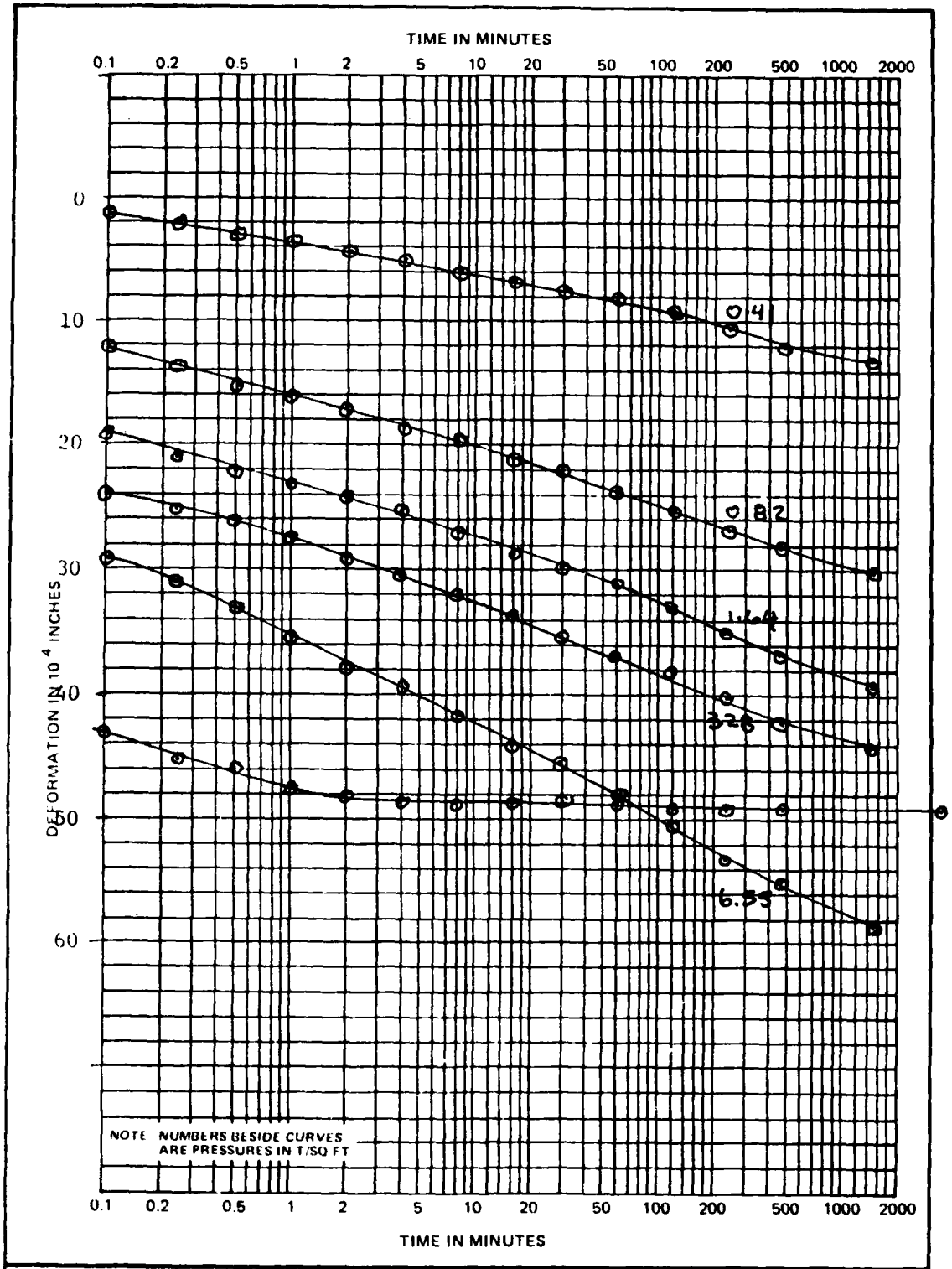
Type of Specimen Remolded		Before Test		After Test	
Diam 4.4 in.	Ht 1.00 in.	Water Content, w_o	13.0 %	w_f	16.6 %
Overburden Pressure, p_o T/sq ft		Void Ratio, e_o	0.572	e_f	0.540
Preconsol. Pressure, p_c T/sq ft		Saturation, S_o	65 %	S_f	88 %
Compression Index, C_c		Dry Density, γ_d	113.1 lb/ft^3		
Classification Clayey Sand(SC-SM)		k_{20} at e_o =	$\times 10^{-}$ cm/sec		
LL 27	G_s 2.85	Project UPPER SANTA ANA RIVER			
PL 20	D_{10}				
Remarks Div. No. 92447		Area U/S Core Borrow			
Remolded to 90% max. at 2%		Boring No. TH-5		Sample No. 16	
above optimum water content.		Depth El 47.5 - 50.0		Date September 1985	

CONSOLIDATION TEST REPORT

ENG FORM 2090 1 MAY 63 PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT)

GPO 936-736

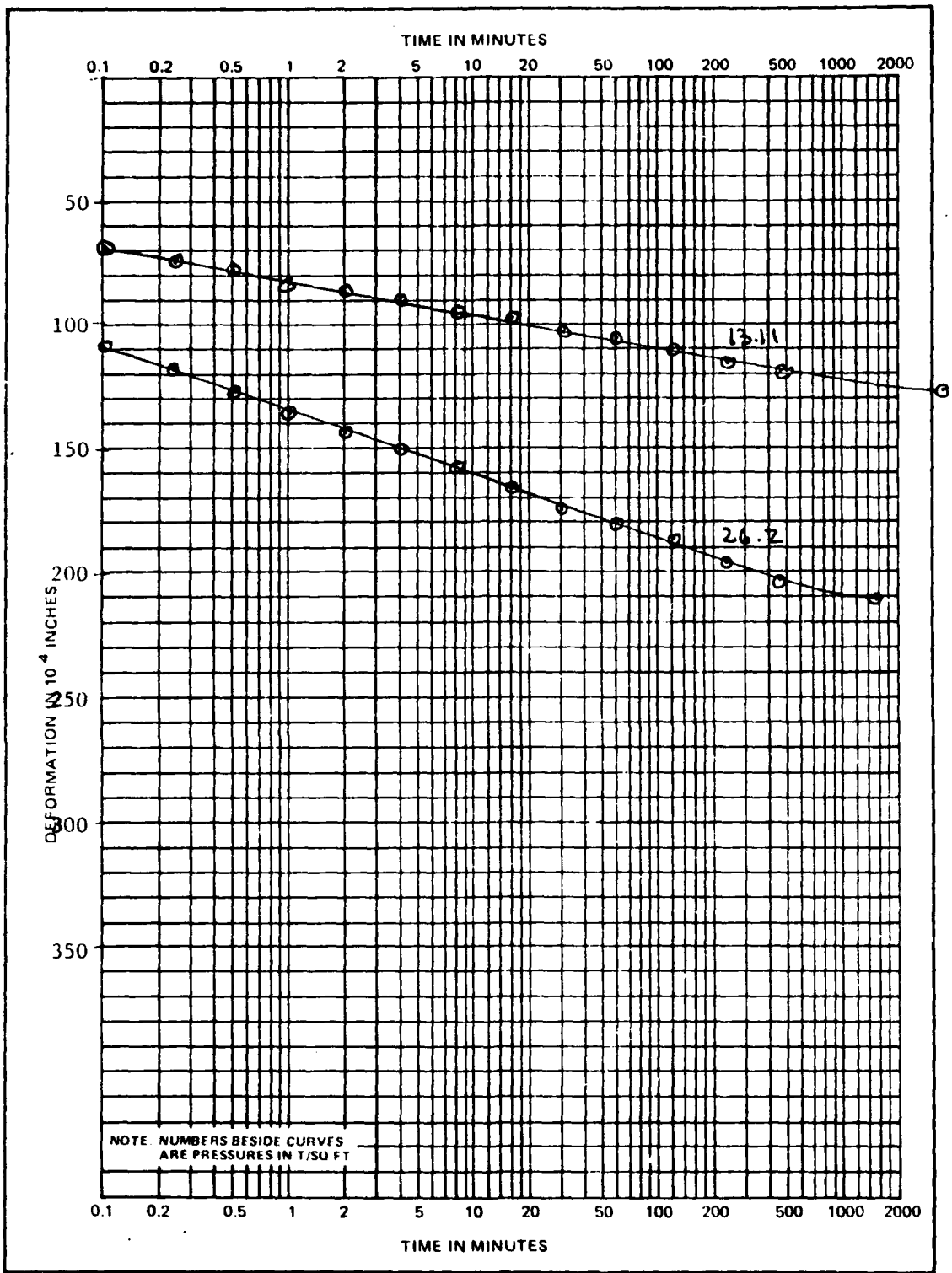
n 3424 plate 28



PROJECT Upper Santa Ana River			
AREA U/S Core Borrow			
BORING NO. TH-5	SAMPLE NO 16	DEPTH EL 47.5 - 50.0	DATE Sept. 1985

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST–TIME CURVES** (TRANSLUCENT)

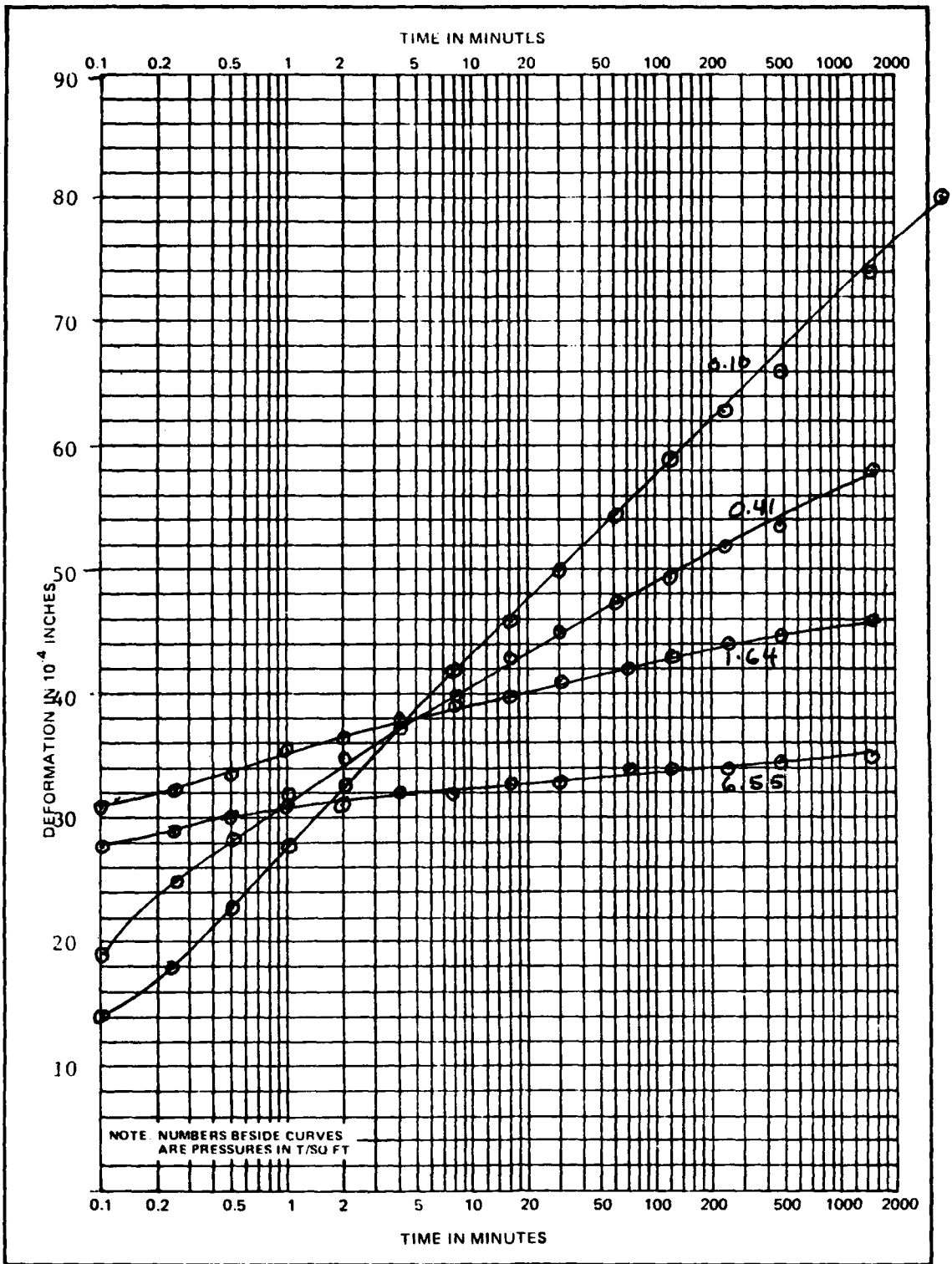
Plate 29



PROJECT Upper Santa Ana River			
AREA U/S Core Borrow			
BORING NO. TH-5	SAMPLE NO 16	DEPTH EL 47.5 - 50.0	DATE Sept. 1985

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST–TIME CURVES** (TRANSLUCENT)

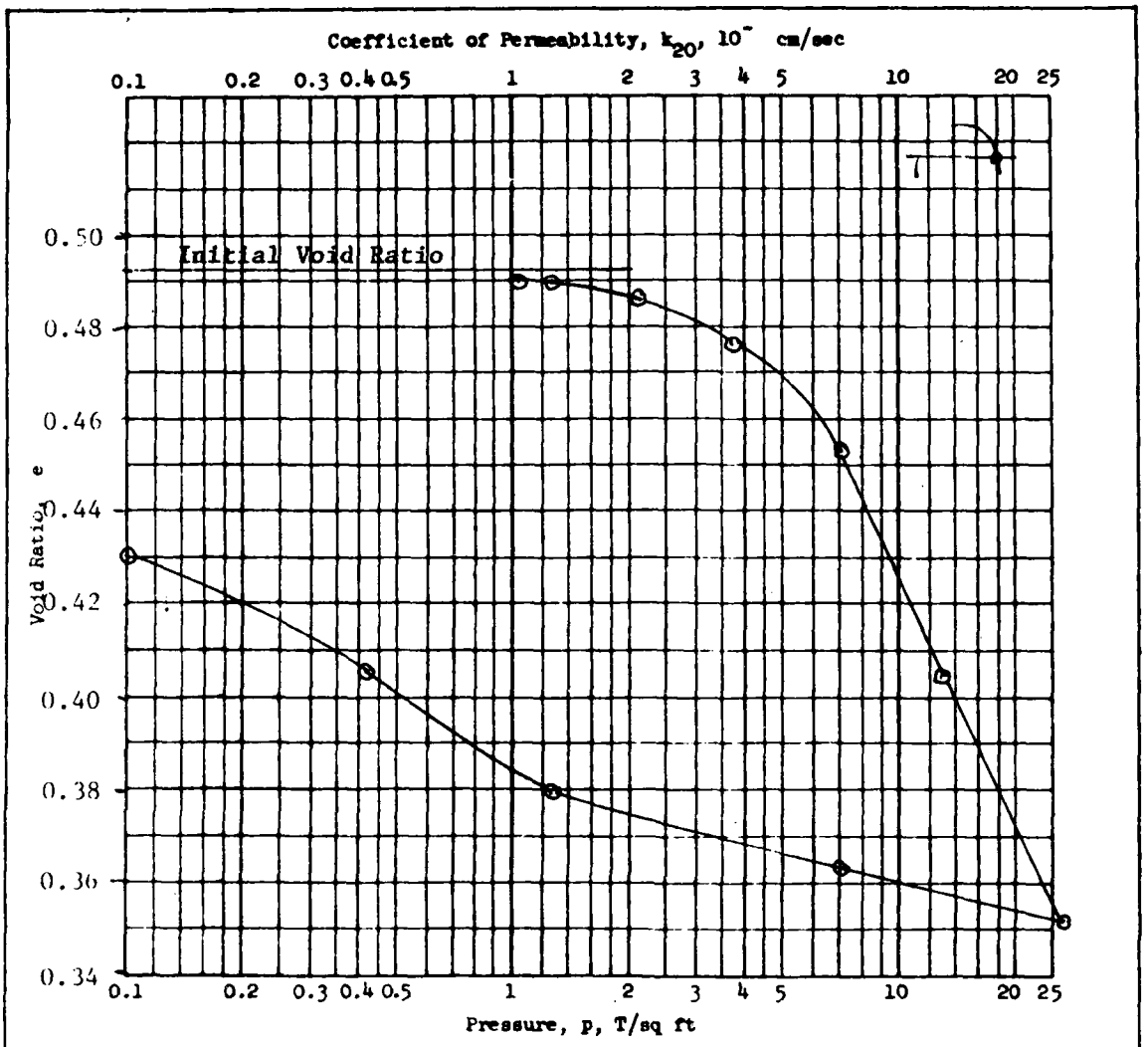
plate 30



PROJECT Upper Santa Ana River			
AREA U/S Core Borrow			
BORING NO. TH-5	SAMPLE NO. 16	DEPTH EL 47.5 - 50.0	DATE Sept. 1985

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST–TIME CURVES** (TRANSLUCENT)

plate 31

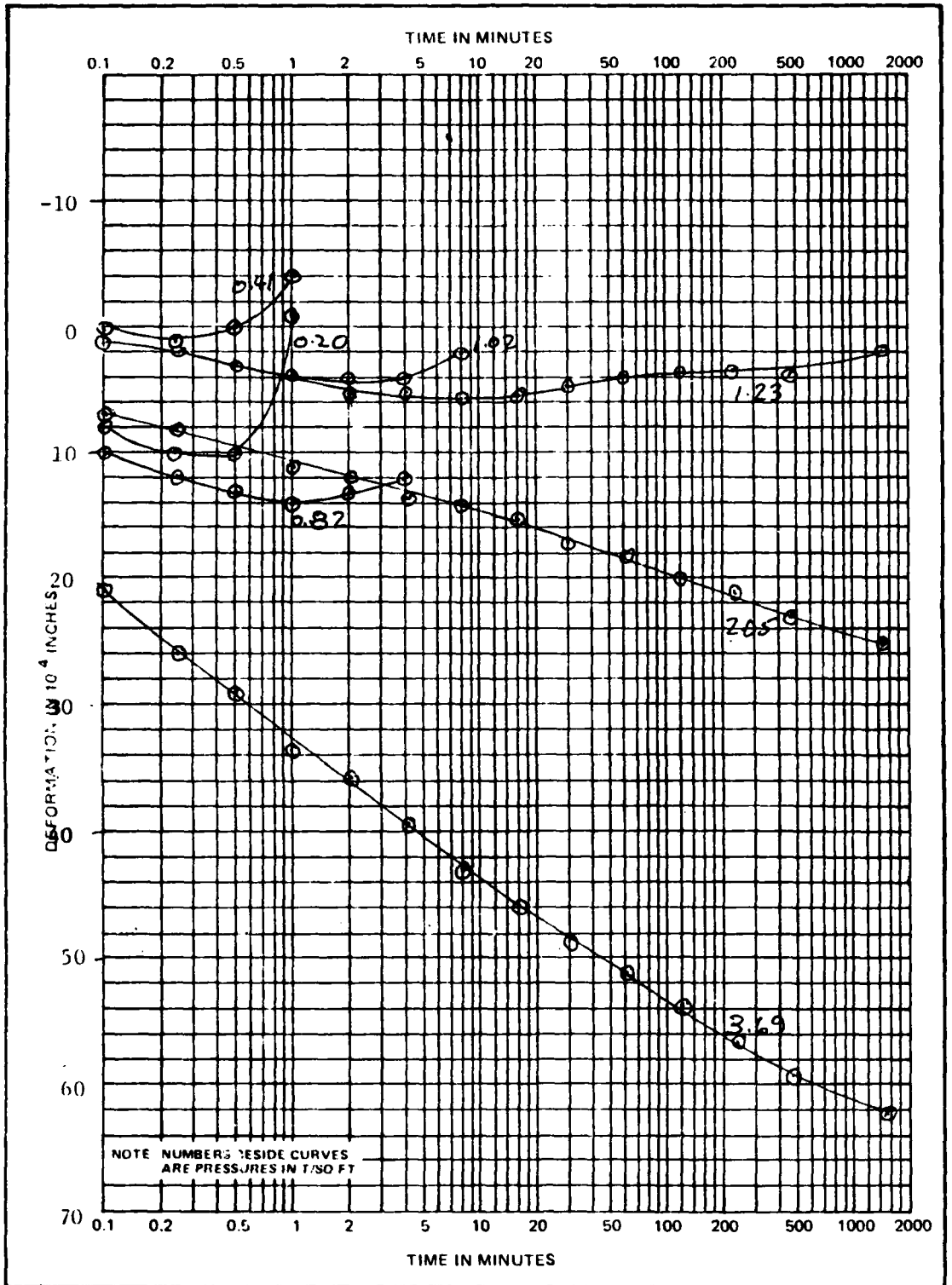


Type of Specimen Remolded			Before Test			After Test	
Diam 4.4 in.	Ht 1.00 in.		Water Content, w_o	12.1 %		w_f	15.9 %
Overburden Pressure, p_o	T/sq ft		Void Ratio, e_o	0.492		e_f	0.455
Preconsol. Pressure, p_c	T/sq ft		Saturation, S_o	68 %		S_f	97 %
Compression Index, C_c			Dry Density, γ_d	115.9 lb/ft^3			
Classification Sandy Clay (CL)			k_{20} at e_o =	x 10^{-} cm/sec			
LL 38	G_s 2.77		Project UPPER SANTA ANA RIVER				
PL 16	D_{10}						
Remarks Div. No. 92450			Area U/S Core Borrow				
Remolded to 90% max. density at			Boring No. TH-7		Sample No. 9		
2% above optimum water content.			Depth El 20-23		Date September 1985		
			CONSOLIDATION TEST REPORT				

ENG FORM 2090 1 MAY 63 PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT)

GPO 936-736

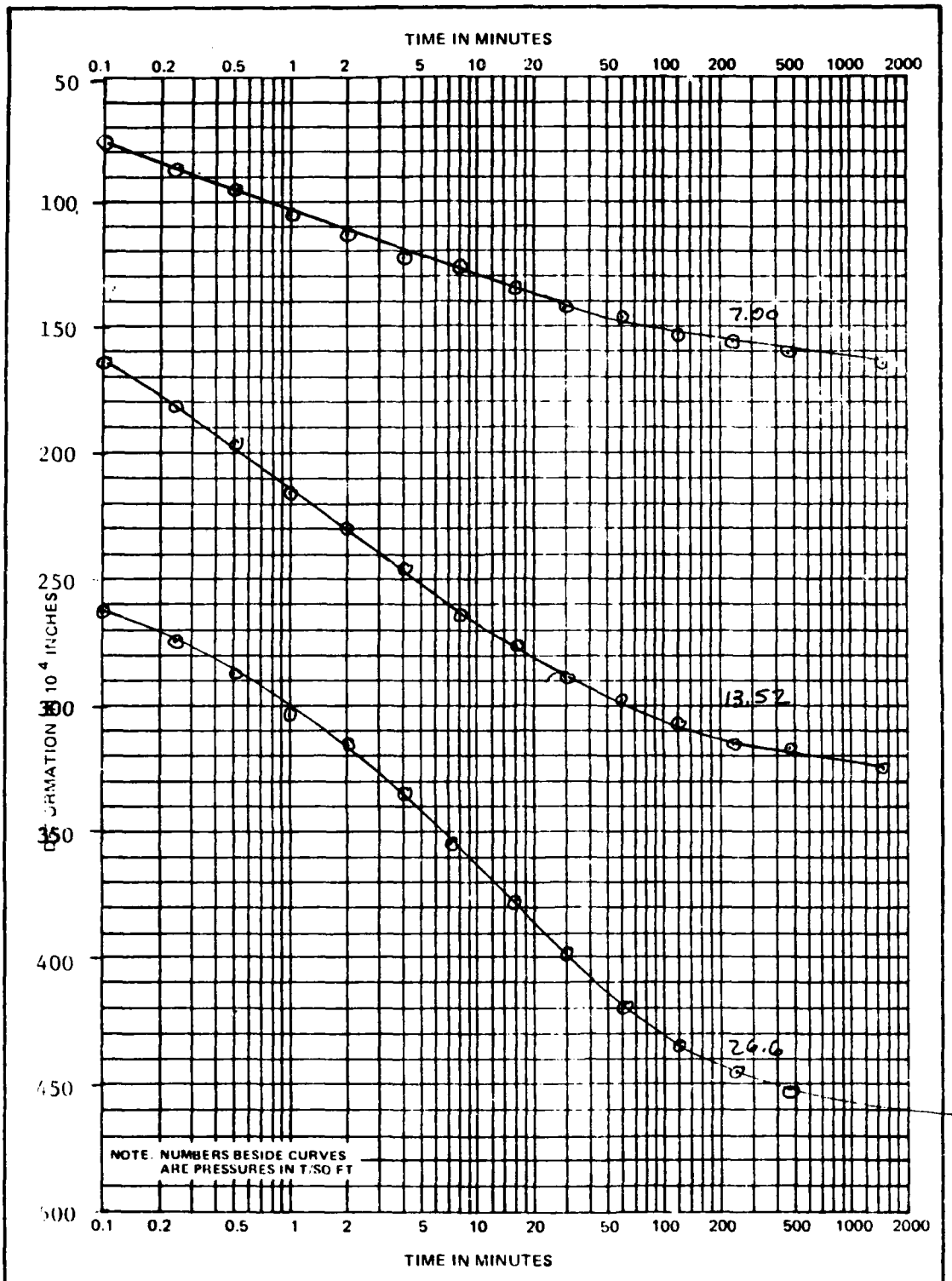
n 3424 plate 32



PROJECT UPPER SANTA ANA RIVER			
AREA U/S Core Borrow			
BORING NO. TH-7	SAMPLE NO 9	DEPTH EL 20-23	DATE Sept. 1985

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST–TIME CURVES** (TRANSLUCENT)

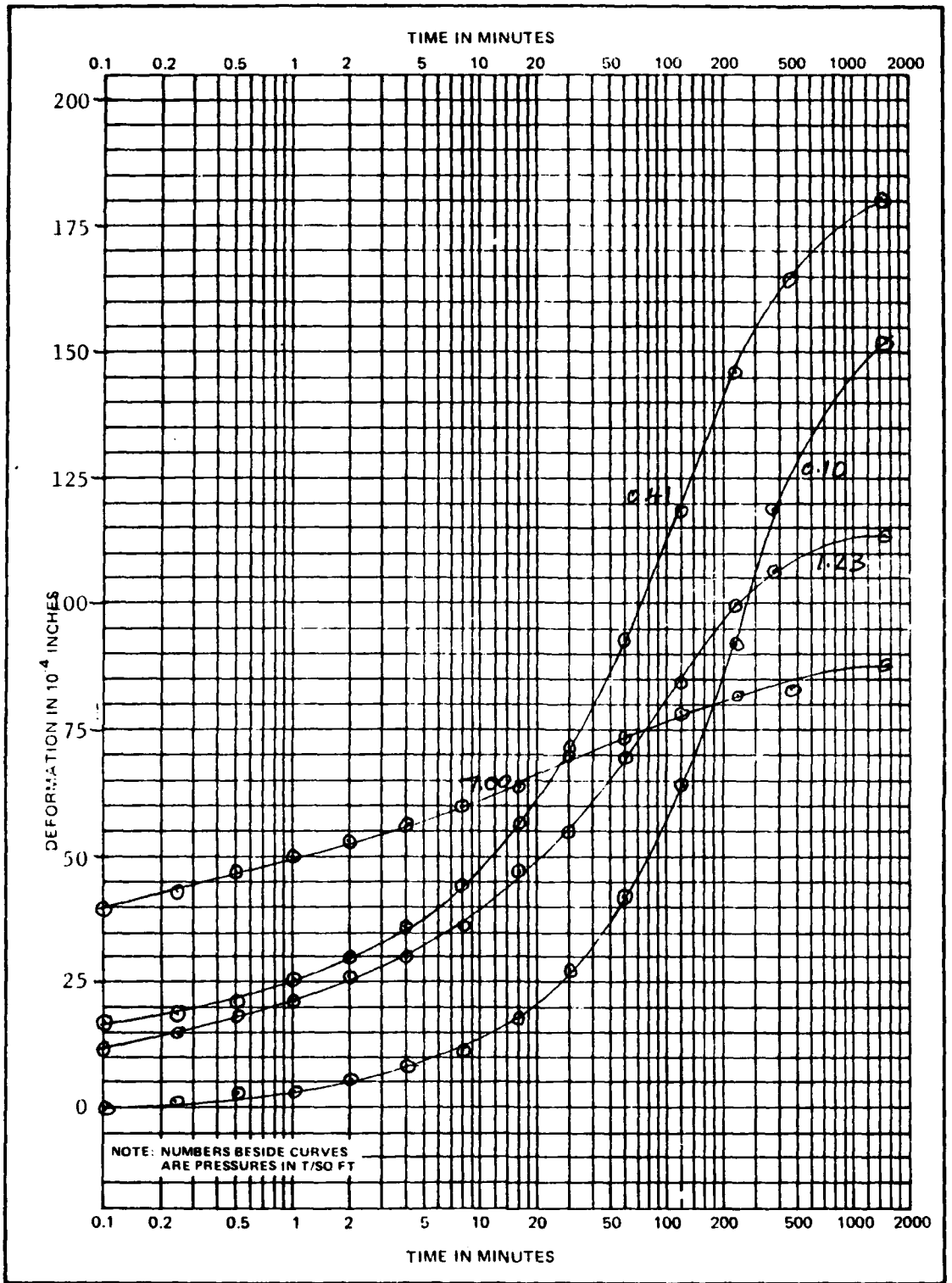
plate 33



PROJECT UPPER SANTA ANA RIVER			
AREA U/S Core Borrow			
BORING NO. TH-7	SAMPLE NO 9	DEPTH EL 20-23	DATE Sept. 1985

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST—TIME CURVES** (TRANSLUCENT)

plate 34



PROJECT UPPER SANTA ANA RIVER			
AREA U/S Core Borrow			
BORING NO. TH-7	SAMPLE NO 9	DEPTH EL 20-23	DATE Sept. 1985

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST--TIME CURVES** (TRANSLUCENT)

plate 35

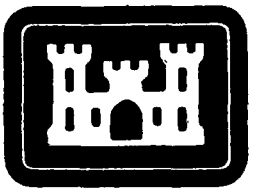
Division Laboratory

Sausalito, California

REPORT
OF
SOIL TESTS

UPPER SANTA ANA RIVER DAM
IMPERVIOUS CORE BORROW MATERIALS

OCTOBER 1986



US Army Corps
of Engineers
South Pacific Division

REPORT
OF
SOIL TESTS

UPPER SANTA ANA RIVER DAM
IMPERVIOUS CORE BORROW MATERIALS

OCTOBER 1986

AUTHORIZATION

1. Results of tests reported herein were requested by the Los Angeles District in laboratory request No. E-86-86-0095 dated 10 July 1986, Change Order No. 1 dated 30 July 1986.

SAMPLES

2. Eleven disturbed sacks of samples were received on 19 May 1986. Identification of samples is shown on the Soil Test Result Summary, plates 1 and 2.

TESTING PROGRAMS

3. The program was in general accordance with the test request. Tests included sieve analysis, Atterberg limits, specific gravity, triaxial compression, compaction, consolidation, and permeability.

TEST METHODS

4. a. Sieve Analysis, Atterberg Limits, Specific Gravity, Compaction, Triaxial Compression, Consolidation, and Permeability. Test methods conformed to the procedure described in Engineer Manual, EM 1110-2-1906, "Laboratory Soil Testing", 30 November 1970.

b. Classification. The soils was classified in accordance with "The Unified Soil Classification System", TM 3-357, Appendix A, April 1980.

RESULTS

5. Results of tests are shown on the following plates:

Subject	Plate No.
Soil Test Result Summary	1-2
Gradation Curves	3-7
Compaction Test Report	8-11
Triaxial Compression Test Report	12-14
Consolidation Test Report	15-49
Permeability	50

U.S. ARMY ENGINEER DIVISION LABORATORY -- SOUTH PACIFIC DIVISION

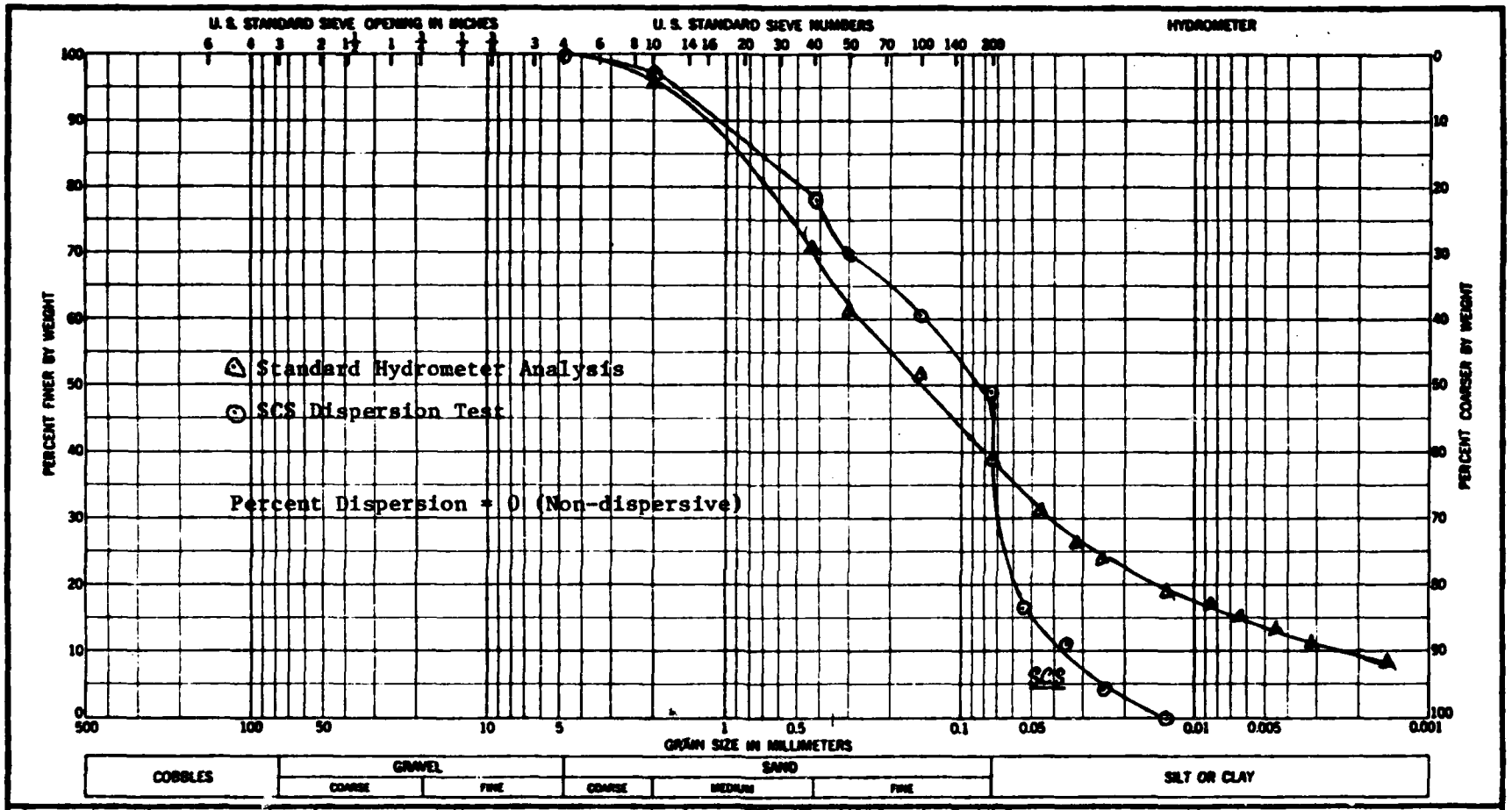
SOIL TEST RESULT SUMMARY

PROJECT Upper Santa Ana River **DATE** July 1986

Division Serial No.	Hole No.	Field Sample No.	Depth Or Elevation From	Depth Or Elevation To	Laboratory Descriptive Classification	Mechanical Analysis-% Finer: Gravel 3"	1½"	¾"	½"	⅜"	Sand #4	#10	#40	#60	#100	Fines #200	Liquid Limit	Plasticity Index	Field Moist %
96586	TH-85-4	A	4		Clayey Sand(SC)	Gs=2.69	100	99	99	98	91	68	57	47	34	27	9	
96587	TH-85-9	"	6																
96588	TH-85-9	B	15		Silty Sand (SM) G= 2.71		100 99	98	97	96	91	82	51	40	31	22		NP	
96589	"	"	26																
96590	TH-85-10	D	18	21	G= 2.69 Silty Sand(SM)	100	99	98	98	97	95	86	54	44	34	25		NP	
96591	TH-85-11	"	20																
96593	TH-85-14	E	19		G= 2.71 Silty Sand (SM)		100	99	98	98	96	91	67	55	45	32		NP	
96594	"	"	44																
96595	TH-21	C	0	13	G=2.71 Clayey Sand(SC)	100 99	98 97	97	96	95	92	85	59	49	40	30	32	14	
96596	TH-23	"	0	15															

PLATE 2

SPD Form 66A
1 May 83



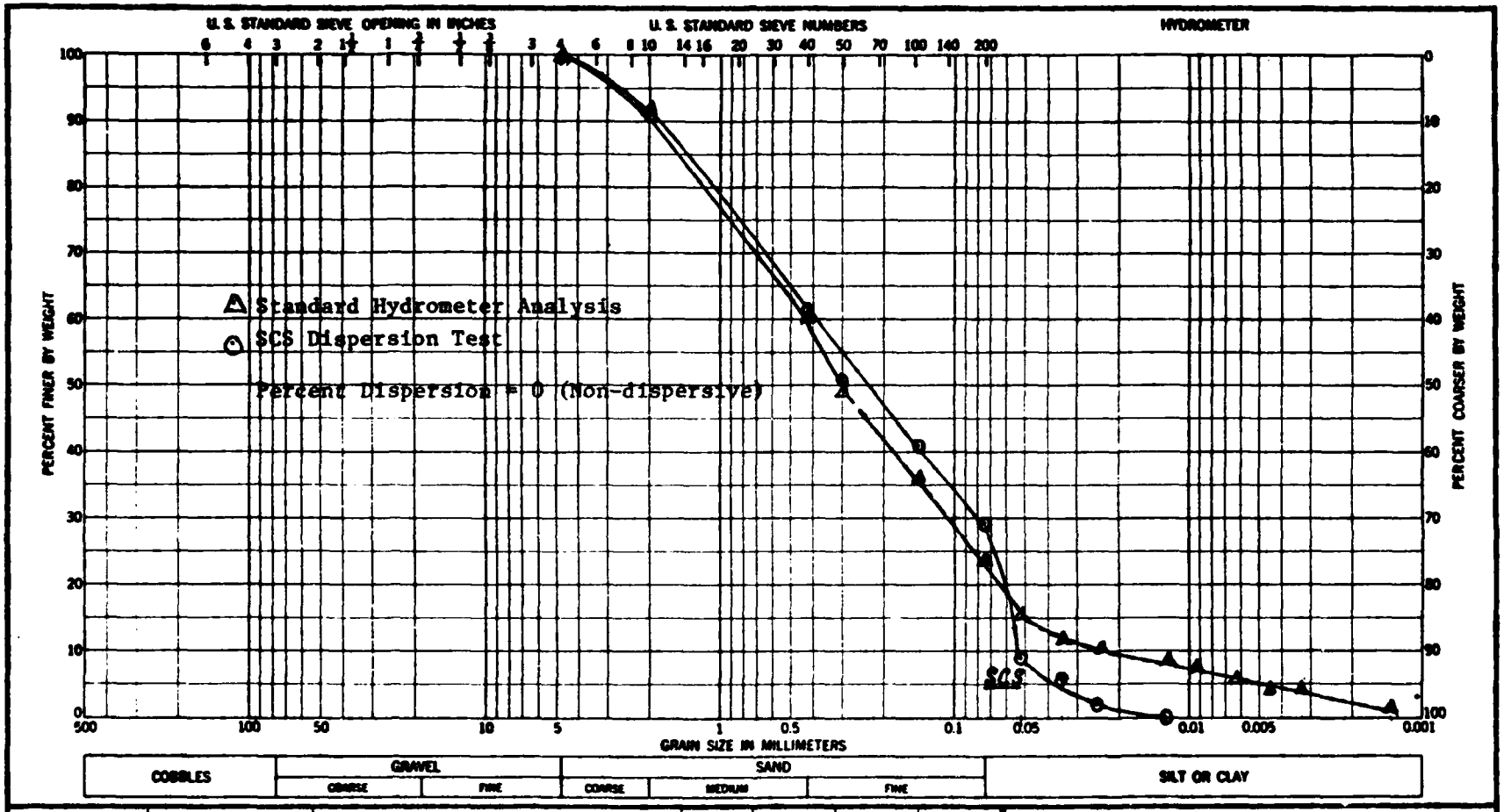
Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
Blend A	4-6	Clayey Sand(SC)		27	18	9

Project: Upper Santa Ana River
Impervious Borrow
Area: Div. No. 96586 - 96587
Boring No.: TH-85- 4/9
Date: September 1986

GRADATION CURVES

PLATE 3

ENG FORM 1 MAY 63 2087



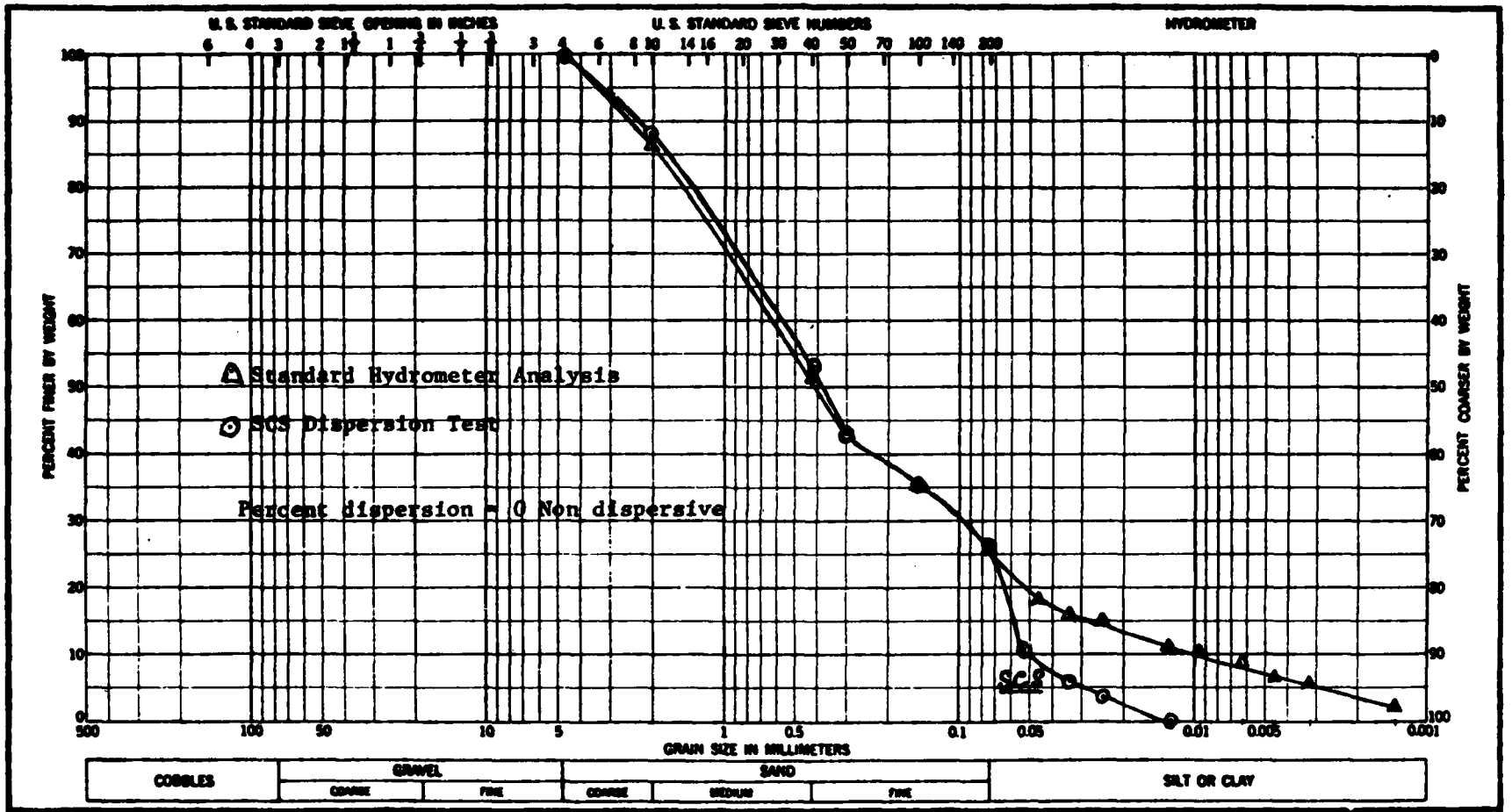
Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
Blend B	15-26	Silty Sand (SM)			NP	

GRADATION CURVES

Project: Upper Santa Ana River
Impervious Borrow
Area: Div. No. 96588 - 96589
Boring No.: TH-85-9
Date: September 1986

PLATE 4

ENG FORM 1 MAY 63 2087



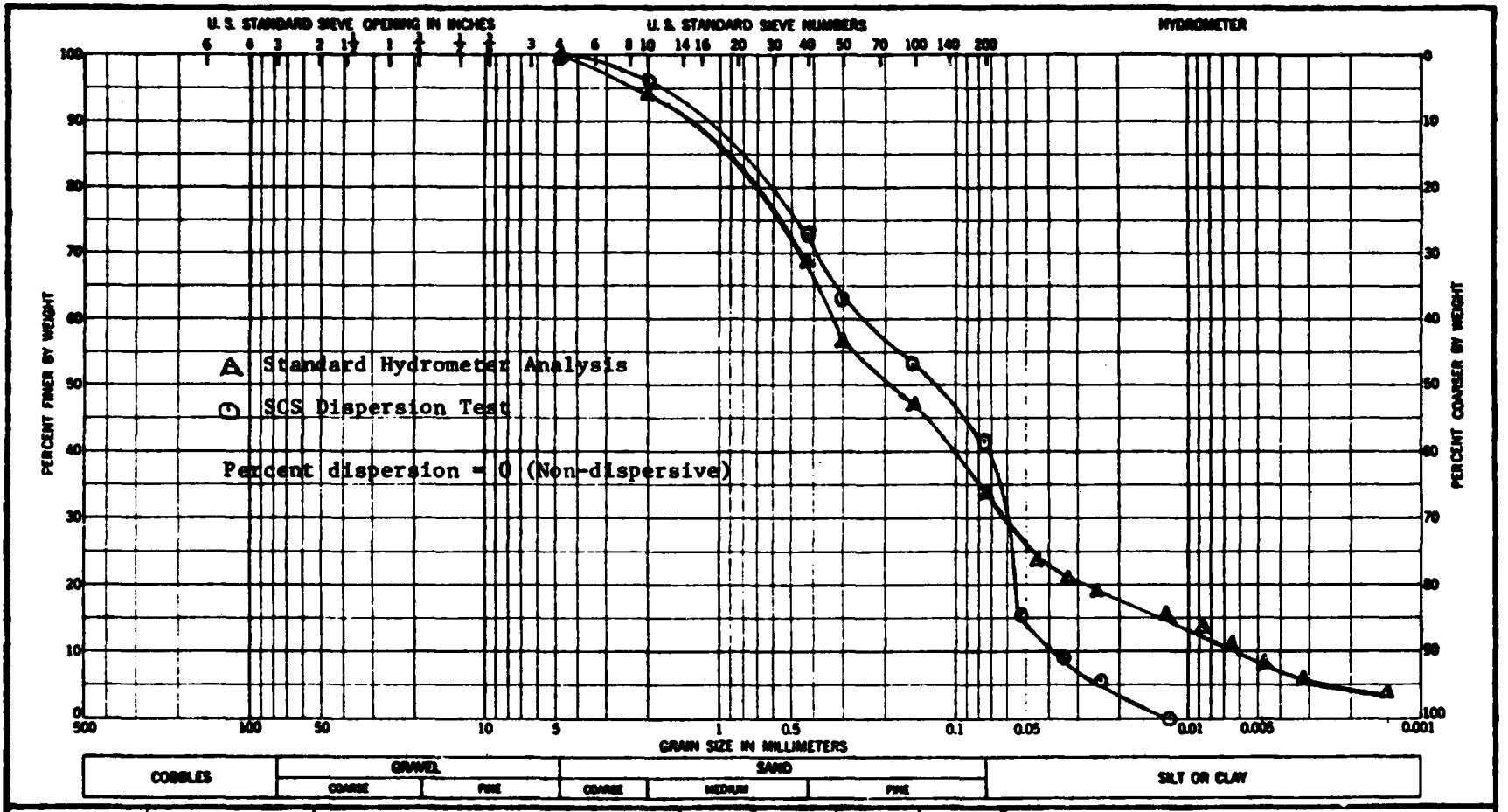
Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
Blend D	18-21	Silty Sand (SM)			NP	

GRADATION CURVES

Project Upper Santa Ana River
Impervious Borrow
Area Div. No. 97590 - 97591
Boring No. TH-85-10, TH-85-11
Date Sept. 1986

PLATE 5

ENG FORM 1 MAY 68 2087



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
Blend E	19-44	Silty Sand (SM)			NP	

GRADATION CURVES

Project Upper Santa Ana River

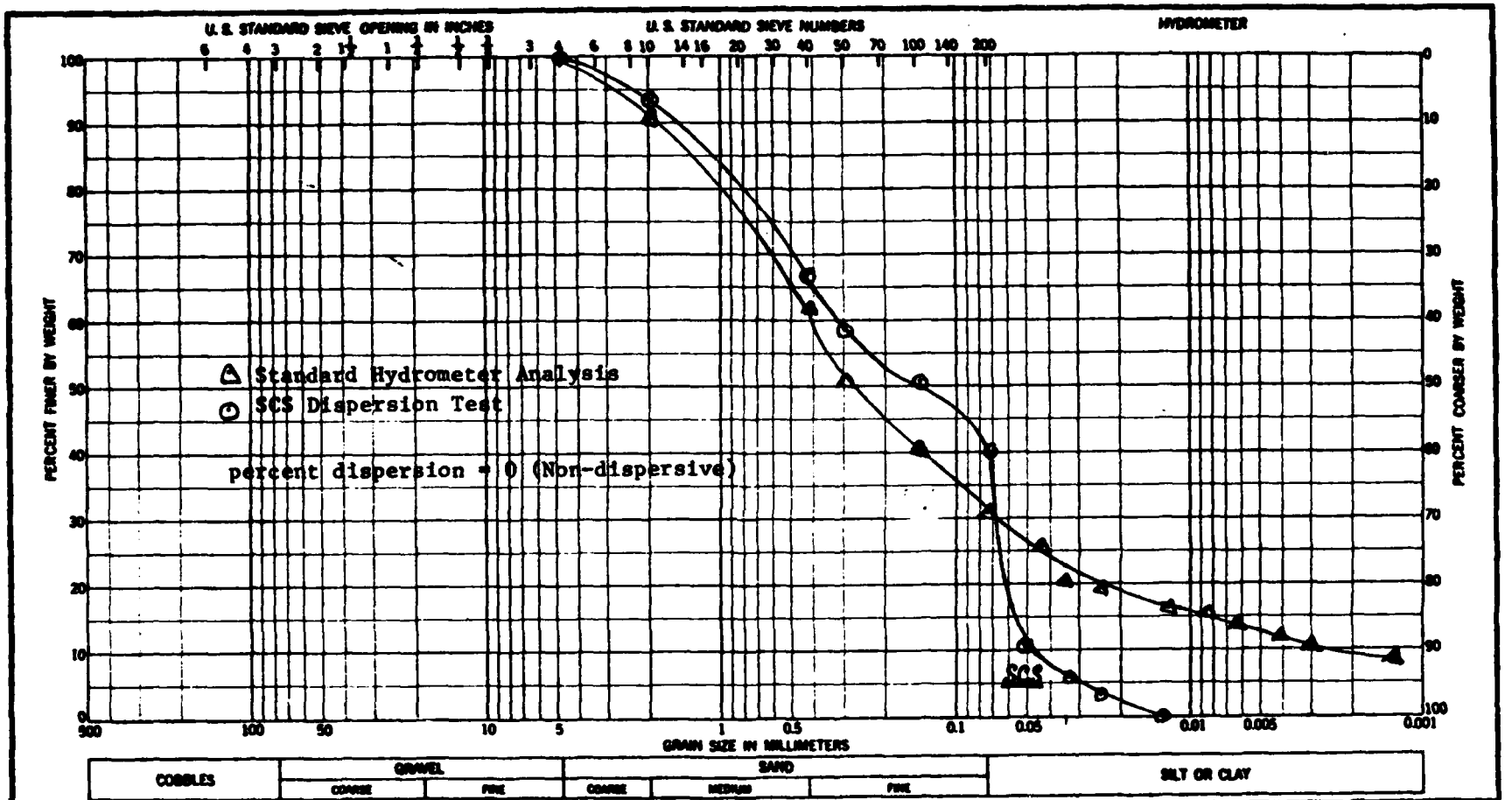
Area Div. No. 96593 - 96594

Boring No. TH-85-14

Date September 1986

PLATE 6

ENG FORM 1 MAY 63 2087



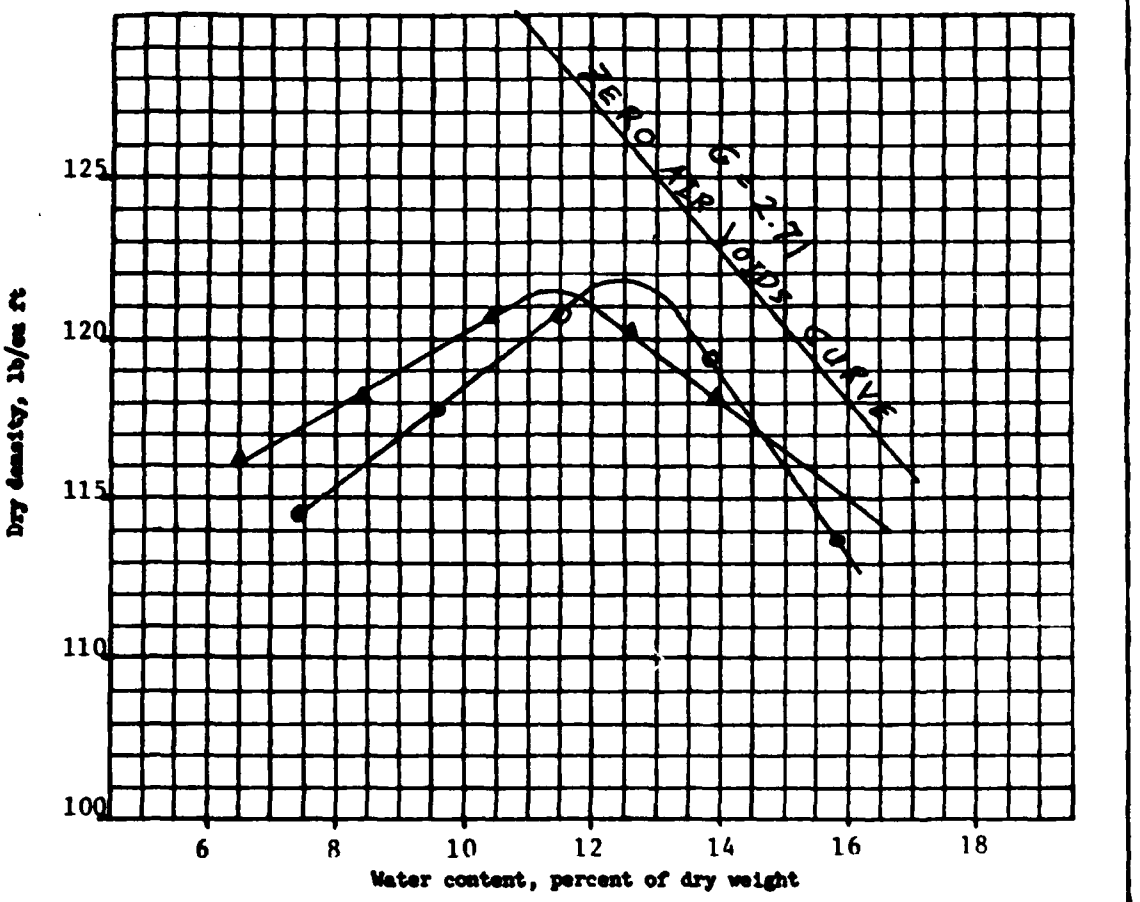
Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
Blend C	0-15	Clayey Sand (SC)		32	18	14

Project: Upper Santa Ana River
Impervious Borrow
Area: Div. No. 96595 - 96596
Boring No.: TH-85-21/23
Date: September 1986

GRADATION CURVES

PLATE 7

ENG FORM 1 MAY 63 2087



Standard compaction test

25 blows per each of 3 layers, with 5 1/2 lb rammer and

12 inch drop. 4 inch diameter mold

Sample No.	Elev or Depth	Classification	G	LL	PL	% > No. 4	% > 3/4 in.
96586	&96587	Clayey Sand (SC)	2.69	27	18	2	0
96588	&96589	Silty Sand (SM)	2.71		NP	9	2

Sample No.	586,587	588,589		
Natural water content, percent	⊙	A		
Optimum water content, percent	12.5	11.4		
Max dry density, lb/cu ft	121.8	121.5		

Remarks		
Div. No.	Hole	Depth
96586	TH-4	NA
96587	TH-9	NA
96588	TH-9	15
96589	TH-9	26

Project: Upper Santa Ana River

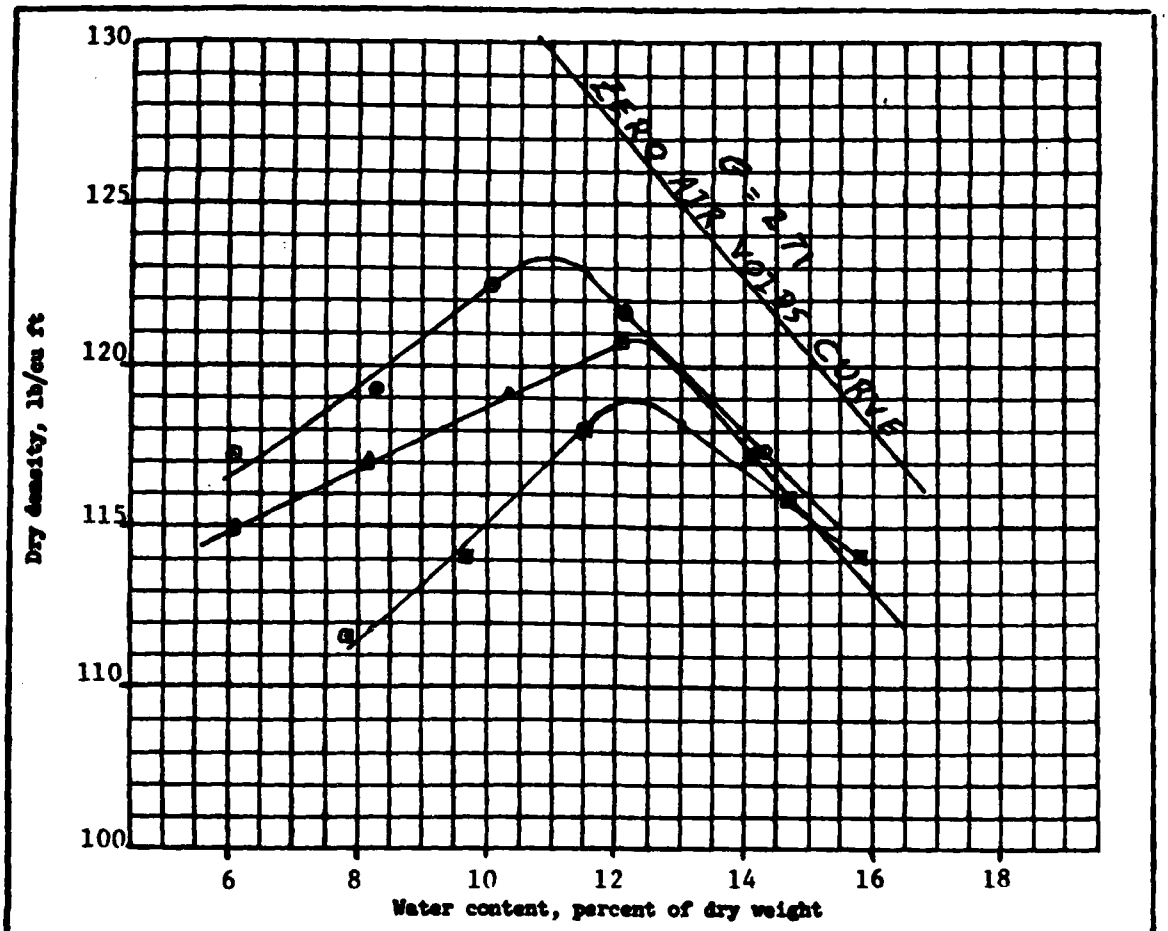
Impervious Core Material

Area

Boring No TH-85-4&9 Date August 1986

COMPACTION TEST REPORT

ENG FORM 1 JUN 65 2091 PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT) PLATE 8



Standard compaction test

25 blows per each of 3 layers, with 5 1/2 lb rammer and

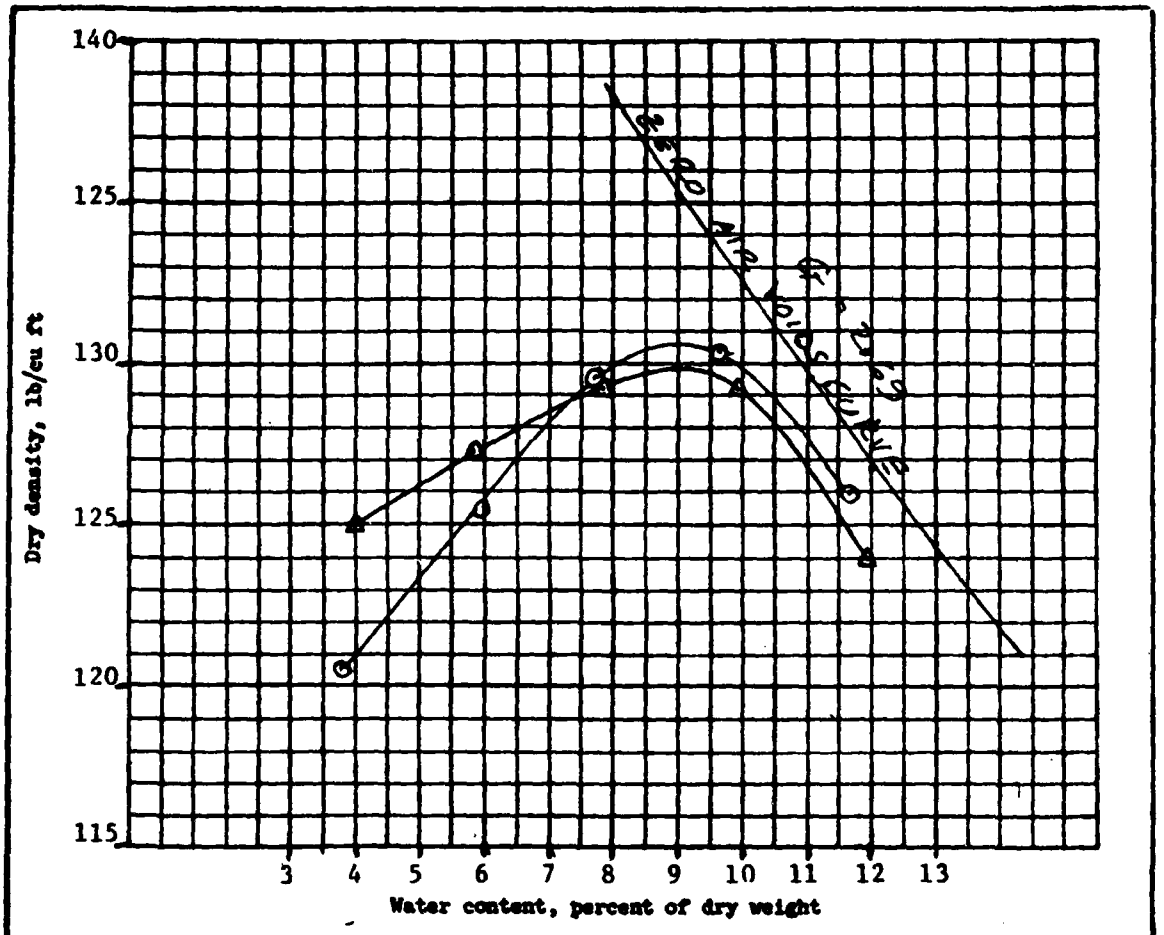
12 inch drop. 4 inch diameter mold

Sample No.	Elev or Depth	Classification	G	LL	PL	% > No. 4	% > 3/4 in.
96590	&96591	Silty Sand (SM)	2.69		NP	5	2
96592	&96594	Silty Sand (SM)	2.71		NP	4	1
96595	&96596	Clayey Sand (SC)	2.71	32	18	8	3

Sample No.	590,591	593,594	595,596	
Natural water content, percent	⊙	△	□	
Optimum water content, percent	10.9	12.3	12.3	
Max dry density, lb/cu ft	123.3	120.8	119.0	

Remarks			Project Upper Santa Ana River	
Div No.	Hole	Depth	Impervious Core Material	
96590 > 96191	TH-10 TH-11	18-20 20	Area	
96593 > 96594	TH-14 TH-14	19 44	Boring No TH-10,11,21&23	Date July 1986
96595 > 96596	TH-21 TH-23	0-13 0-15	**COMPACTION TEST REPORT**	

ENG FORM 1 JUN 65 2091 PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT) PLATE 9



Modified compaction test

25 blows per each of 5 layers, with 10 lb rammer and

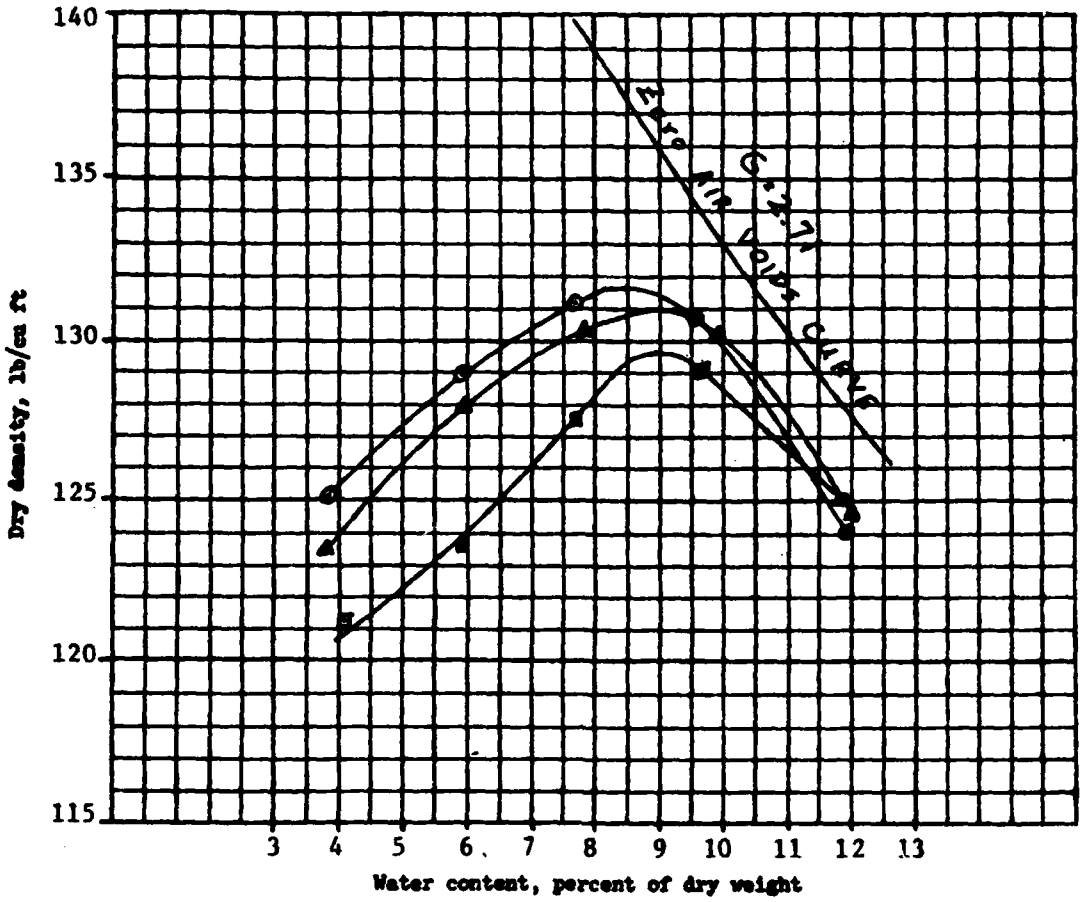
18 inch drop. 4 inch diameter mold

Sample No.	Elev or Depth	Classification	G	LL	PL	% > No. 4	% > 3/4 in.
96586	96587	Clayey Sand (SC)	2.69	27	18	2	0
96588	96589	Silty Sand (SM)	2.71		NP	9	2

Sample No.	⊙	△		
Natural water content, percent				
Optimum water content, percent	9.0	9.0		
Max dry density, lb/cu ft	130.6	129.8		

Remarks	Project Upper Santa Ana River	
Div. No. Hole No. 96586 TH-85-4 96587 ⊙ TH-85-9 96588 TH-85-9 96589 △ "	Impervious Core Borrow Material Area	
	Boring No. TH-85-4 & 9	Date August 1986
	COMPACTION TEST REPORT	

ENG FORM 1 JUN 85 2091 PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT) PLATE 10



Modified compaction test

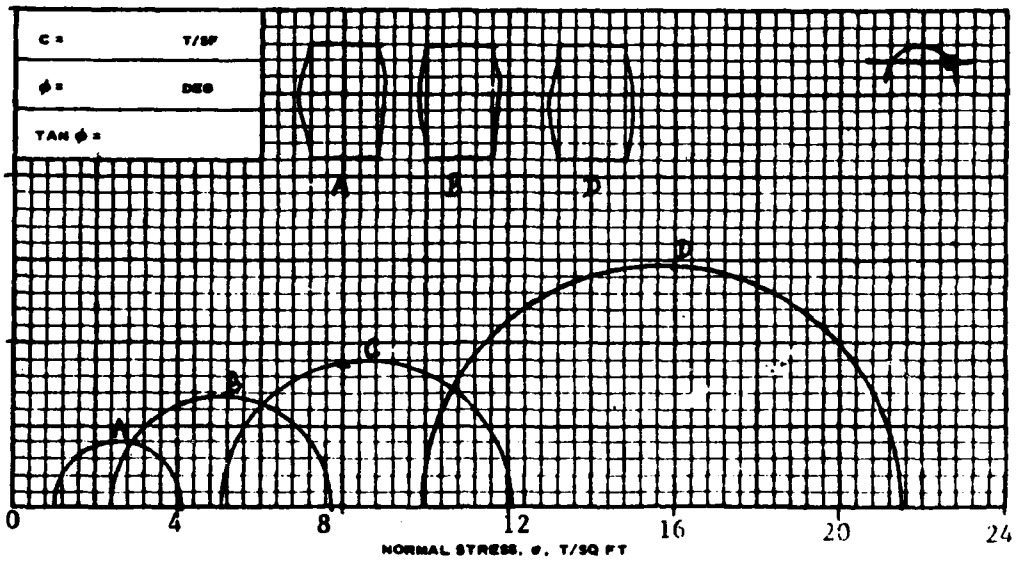
25 blows per each of 5 layers, with 10 lb rammer and 18 inch drop. 4 inch diameter mold

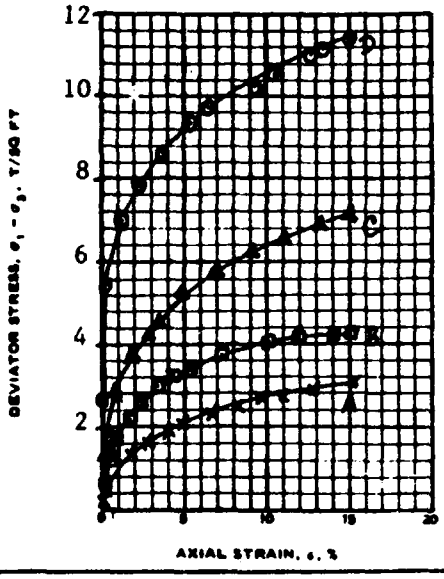
Sample No.	Elev or Depth	Classification	G	LL	PL	% > No. 4	% > 3/4 in.
96590,	96591	Silty Sand(SM)	2.69		NP	5	2
96593,	96594	Silty Sand(SM)	2.71		NP	4	1
96595,	96596	Clayey Sand(SC)	2.71	32	18	8	3

Sample No.	⊙	△	□	
Natural water content, percent	8.4	9.0	9.0	
Optimum water content, percent	131.6	130.9	129.6	
Max dry density, lb/cu ft				

Remarks			Project	Upper Santa Ana River Dam
Div. No.		Hole No.		Impervious Core Borrow Materials
96590 96591	⊙	TH-85-10 TH-85-11	Area	
96593 96594	△	TH-85-14 TH-85-14	Boring No. TH-85-10&11, 14, 21 & 23	Date August 1986
96595 96596	□	TH-85-21 TH-85-23	**COMPACTION TEST REPORT**	

ENG FORM 1 JUN 69 2091 PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT) PLATE 11





SPECIMEN NO.			A	B	C	D
INITIAL	WATER CONTENT, %	w_o	13.8	14.0	14.3	13.7
	DRY DENSITY LB/CU FT	γd_o	119.6	119.4	119.1	119.6
	SATURATION, %	S_o	92	93	94	92
	VOID RATIO	e_o	0.404	0.406	0.410	0.40
BEFORE SHEAR	WATER CONTENT, %	w_c	14.5	14.5	13.7	12.6
	DRY DENSITY LB/CU FT	γd_c	120.7	120.9	122.8	125.4
	SATURATION, %	S_c	100	100	100	100
	VOID RATIO	e_c	0.391	0.389	0.367	0.339
	FINAL BACK PRESSURE, T/SQ FT	u_b	7.2	7.2	7.2	7.2
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	1.00	2.50	5.00	10.00
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{max}$	3.08	4.26	7.06	11.52
TIME TO $(\sigma_1 - \sigma_3)_{max}$, MIN		t_f	336	306	309	378
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ult}$	–	–	–	–
INITIAL DIAMETER, IN.		D_o	2.80	2.80	2.80	2.80
INITIAL HEIGHT, IN.		H_o	6.45	6.45	6.16	6.45

CONTROLLED- TEST

DESCRIPTION OF SPECIMENS Clayey Sand (SC)

LL 27 | PL 18 | PI 9 | G_s 2.69

TYPE OF SPECIMEN Remolded | TYPE OF TEST R

REMARKS: 98% max at 1.5% above Optimum W.C.

Div. No. 96586
96587 Blend "A".

PROJECT Upper Santa Ana River

BORING NO. TH-85-4 ; 9 | SAMPLE NO. Blend "A".

DEPTH/ELEV 4' and 6'

LABORATORY SPDL | DATE October 1986

TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT (EM 1110-2-1906) PLATE 12

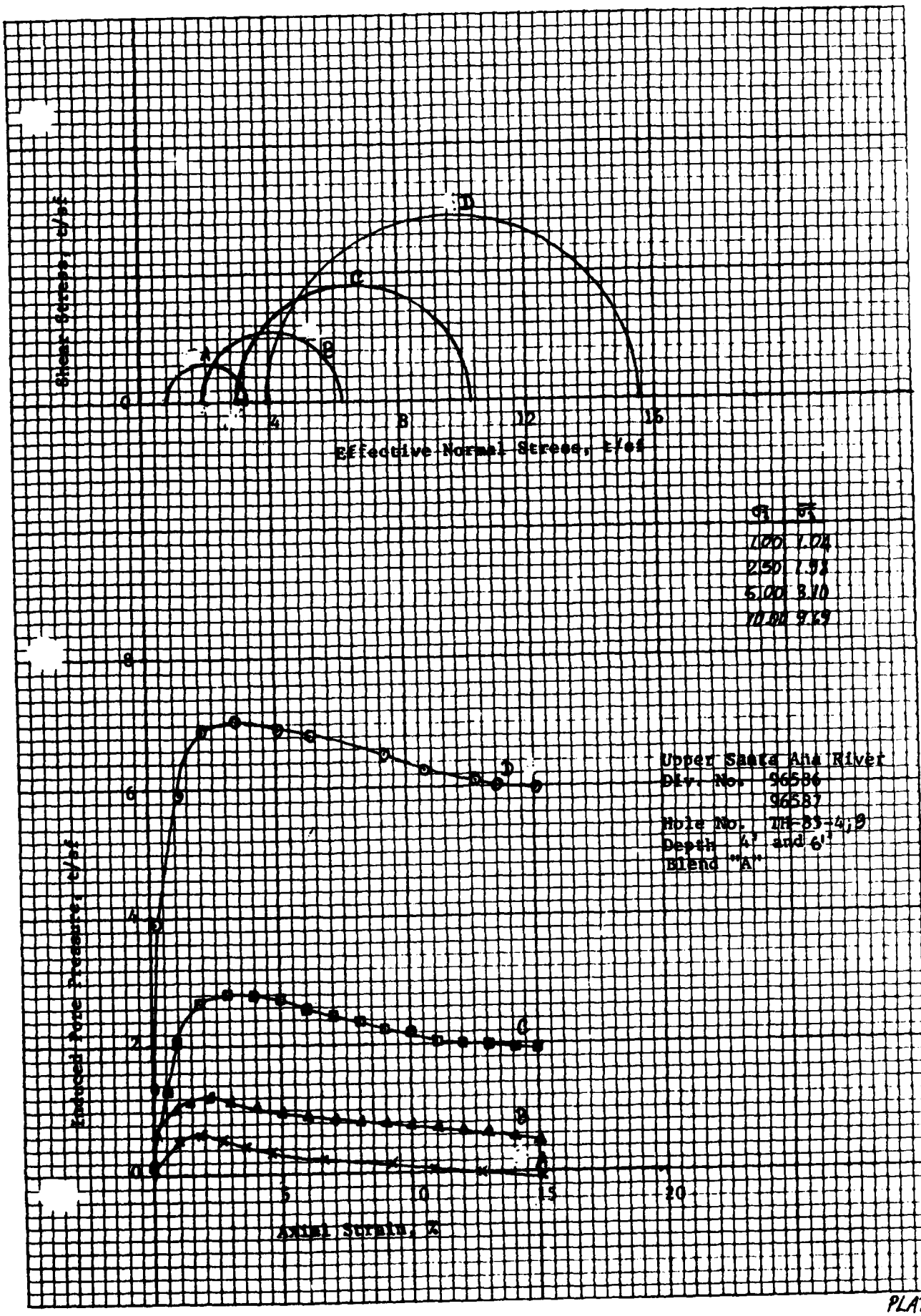
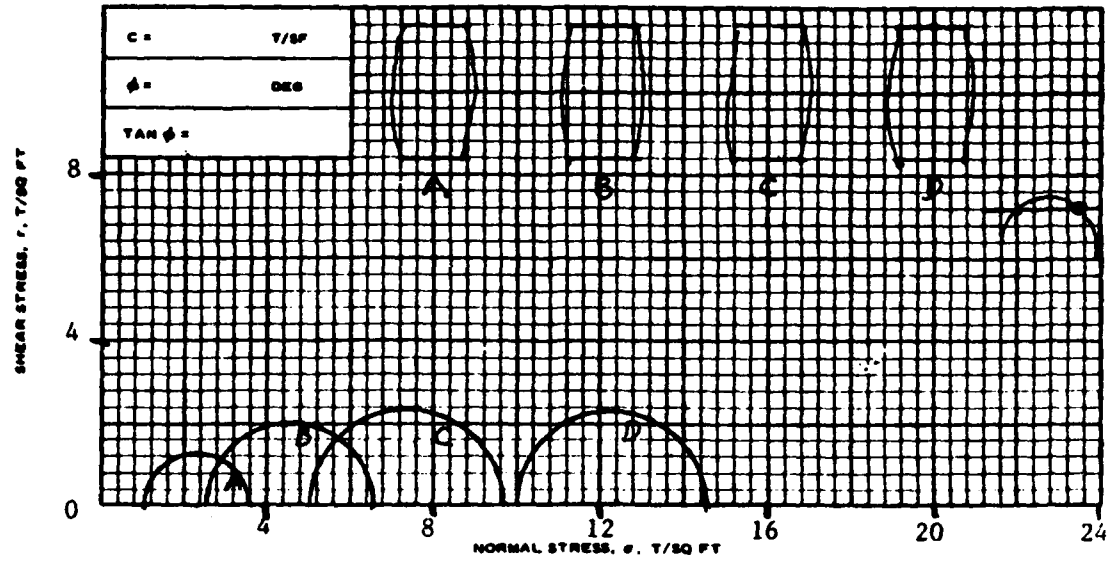
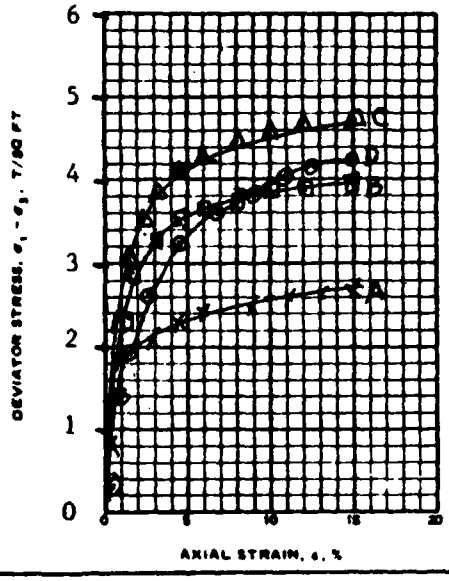

Shear Stress, t/sf
Effective Normal Stress, t/sf
0
4
8
12
16
A
B
C
D
σ1 σ3
1.00 1.04
2.50 1.93
5.00 3.10
10.00 6.29
Upper Santa Ana River
Div. No. 96586
96587
Hole No. UR-B3-4,9
Depth 4' and 6'
Blend "A"
Deviator Stress, t/sf
Axial Strain, %
0
2
4
6
8
5
10
15
20
a
b
c
d

PLATE 13





SPECIMEN NO.			A	B	C	D
INITIAL	WATER CONTENT, %	w_o	14.2	14.1	14.1	14.7
	DRY DENSITY LB/CU FT	γd_o	116.7	116.8	116.8	116.2
	SATURATION, %	S_o	86	85	85	87
	VOID RATIO	e_o	0.449	0.448	0.448	0.456
BEFORE SHEAR	WATER CONTENT, %	w_c	14.2	14.1	14.1	14.5
	DRY DENSITY LB/CU FT	γd_c	117.2	117.8	119.3	118.6
	SATURATION, %	S_c	87	88	91	92
	VOID RATIO	e_c	0.443	0.435	0.421	0.426
	FINAL BACK PRESSURE, T/SQ FT	u_b	-	-	-	-
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	1.00	2.50	5.00	10.00
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	2.64	4.00	4.79	4.31
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	16	15	15	15.5
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$	-	-	-	-
INITIAL DIAMETER, IN.		D_o	2.80	2.80	2.80	2.80
INITIAL HEIGHT, IN.		H_o	6.45	6.45	6.45	6.45

CONTROLLED- Strain TEST

DESCRIPTION OF SPECIMENS Clayey Sand (SC)

LL 32	PL 18	PI 14	G_s 2.71	TYPE OF SPECIMEN Remolded	TYPE OF TEST Q

REMARKS: Remolded to 98% max. density at 2% above optimum water content.

Div. No. 96586 & 96595

PROJECT Upper Santa Ana River

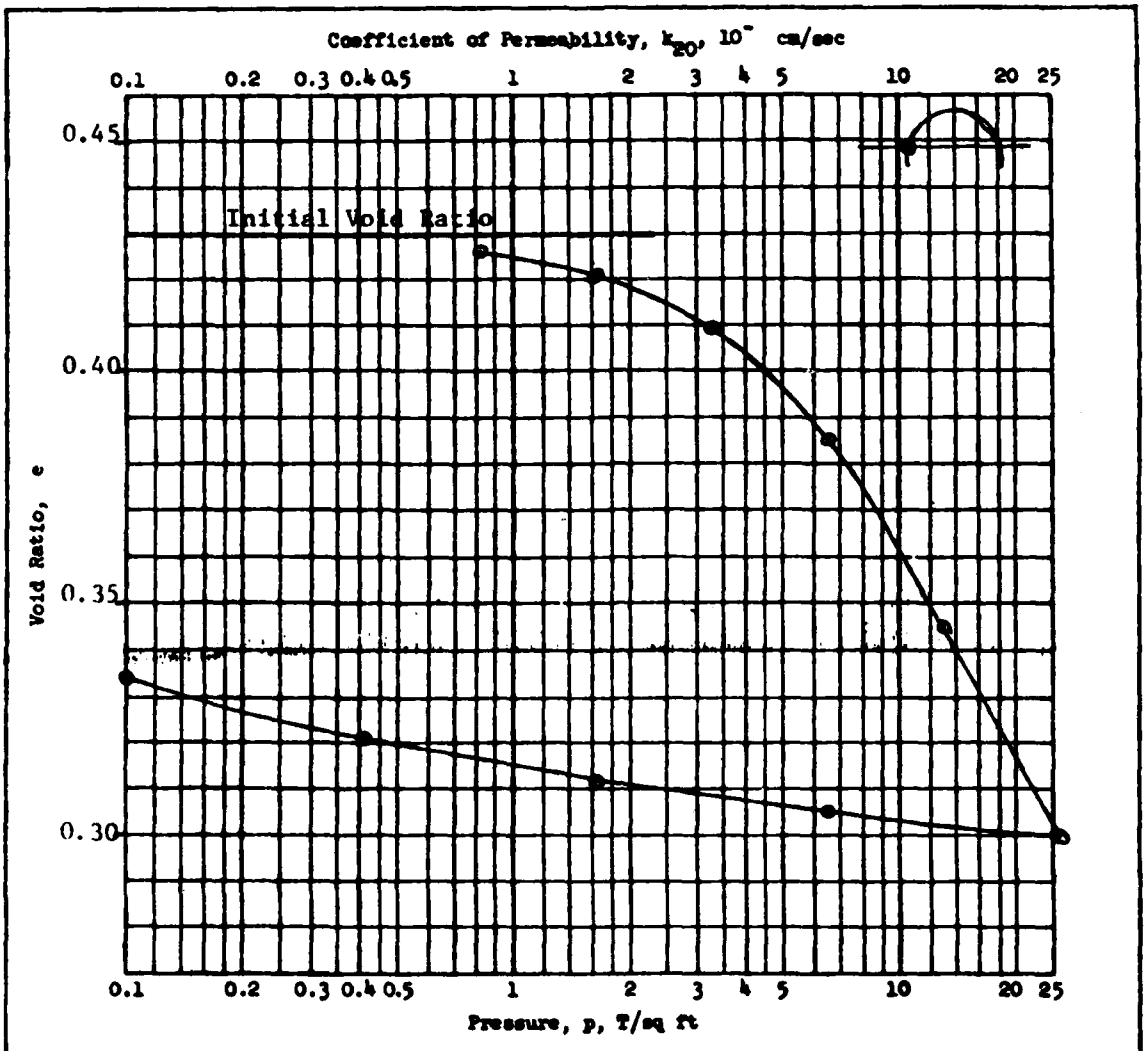
BORING NO. TH-85-21&23 SAMPLE NO. Blend "C"

DEPTH/ELEV

LABORATORY SPDL DATE September 1986

TRIAXIAL COMPRESSION TEST REPORT

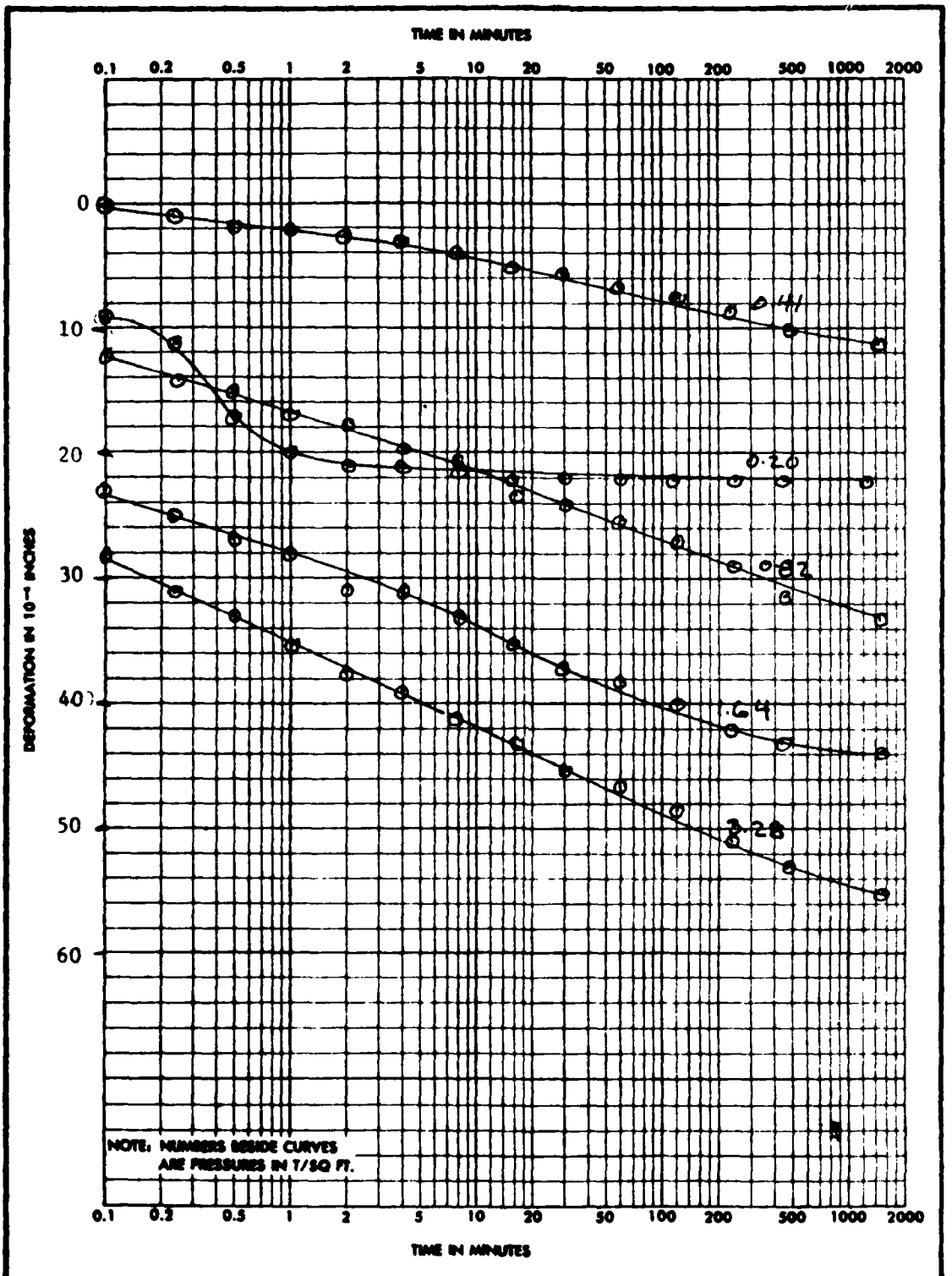
ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE TRANSLUCENT (EM 1110-2-1906) PLATE 14



Type of Specimen Remolded		Before Test		After Test	
Diam 4.4 in.	Ht 1.00 in.	Water Content, w_o	6.1 %	w_f	13.5 %
Overburden Pressure, p_o T/sq ft		Void Ratio, e_o	0.430	e_f	0.367
Preconsol. Pressure, p_c T/sq ft		Saturation, S_o	38.2 %	S_f	98.7 %
Compression Index, C_c		Dry Density, γ_d	117.4 lb/ft^3		
Classification Clayey Sand(SC)		k_{20} at e_o =	x 10^{-} cm/sec		
LL 27	G_s 2.69	Project Upper Santa Ana River Dam			
PL 18	D_{10}	Impervious Core Material			
Remarks Div. No. 96586 & 96587		Area			
Remolded to 90% max. mod.		Boring No. TH-85-469		Sample No.	
density at 3% below optimum		Depth El 4' & 6'		Date August 1986	
water content		CONSOLIDATION TEST REPORT			

ENG FORM 1 MAY 85 2090 PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT) GPO 936-736

n 3424 PLATE 15

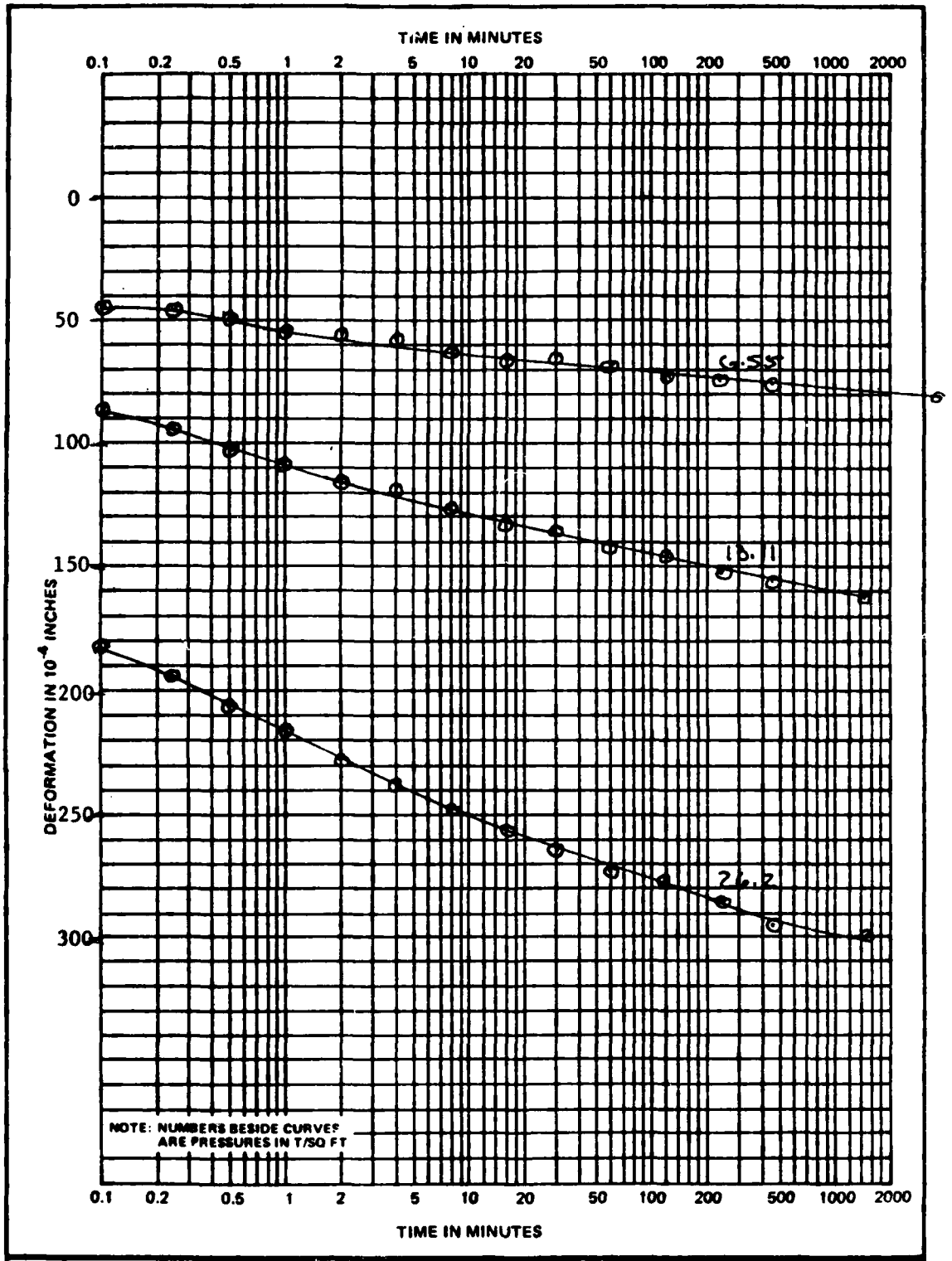


PROJECT	Upper Santa Ana River			
AREA	Div. No. 96586 & 96587			
BORING NO.	TH-4 & 9	SAMPLE NO. Comp A	DEPTH EL 4-6	DATE August 1986

ENG FORM 2088 1 MAY 63 PREVIOUS EDITIONS ARE OBSOLETE.

CONSOLIDATION TEST—TIME CURVES (TRANSLUCENT)

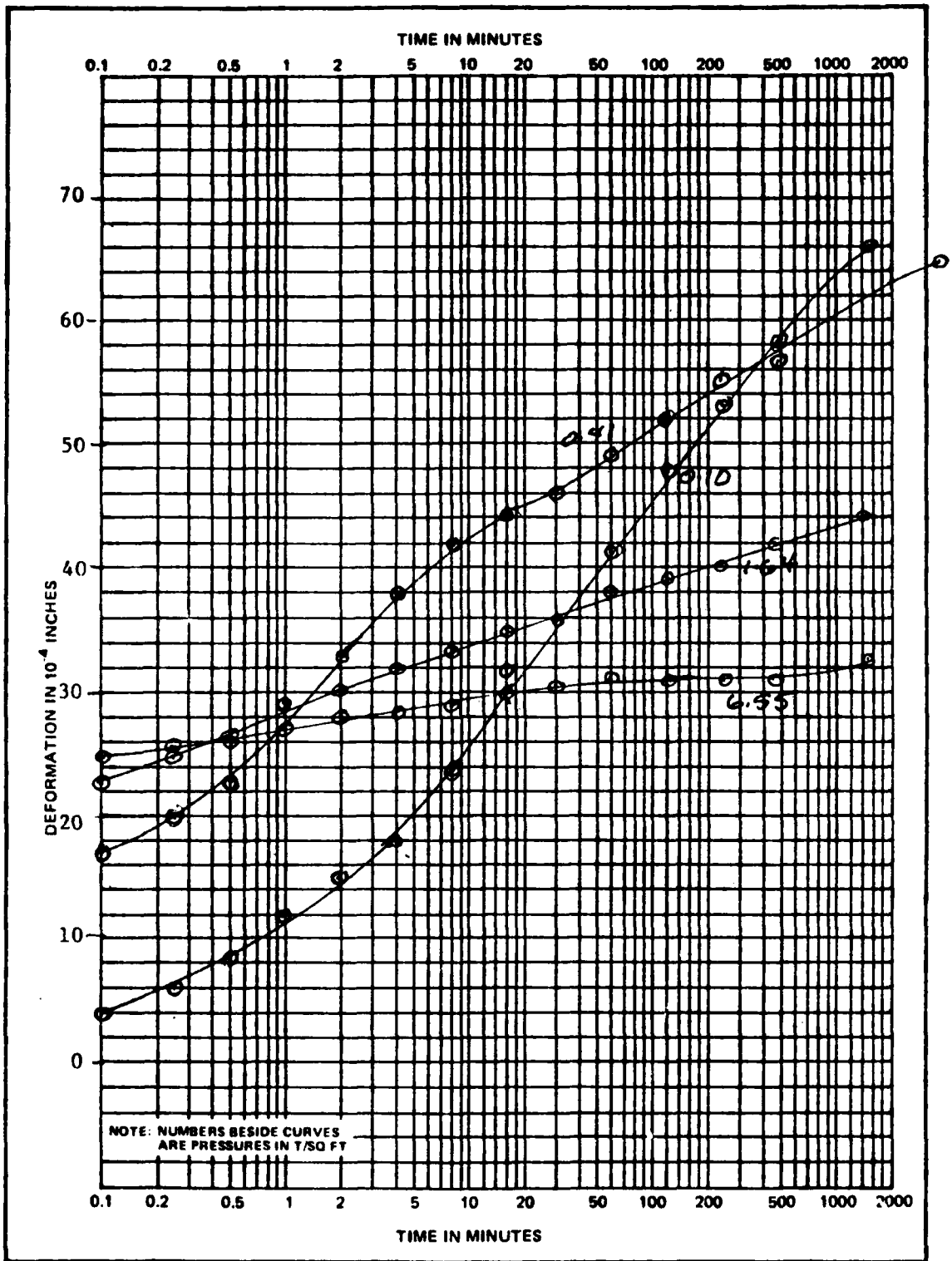
PLATE 16



PROJECT	Upper Santa Ana River			
AREA	Div. No. 96586 & 96587			
BORING NO.	TH 4 & 9	SAMPLE NO Comp A	DEPTH EL 4-6	DATE August 1986

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST–TIME CURVES** (TRANSLUCENT)

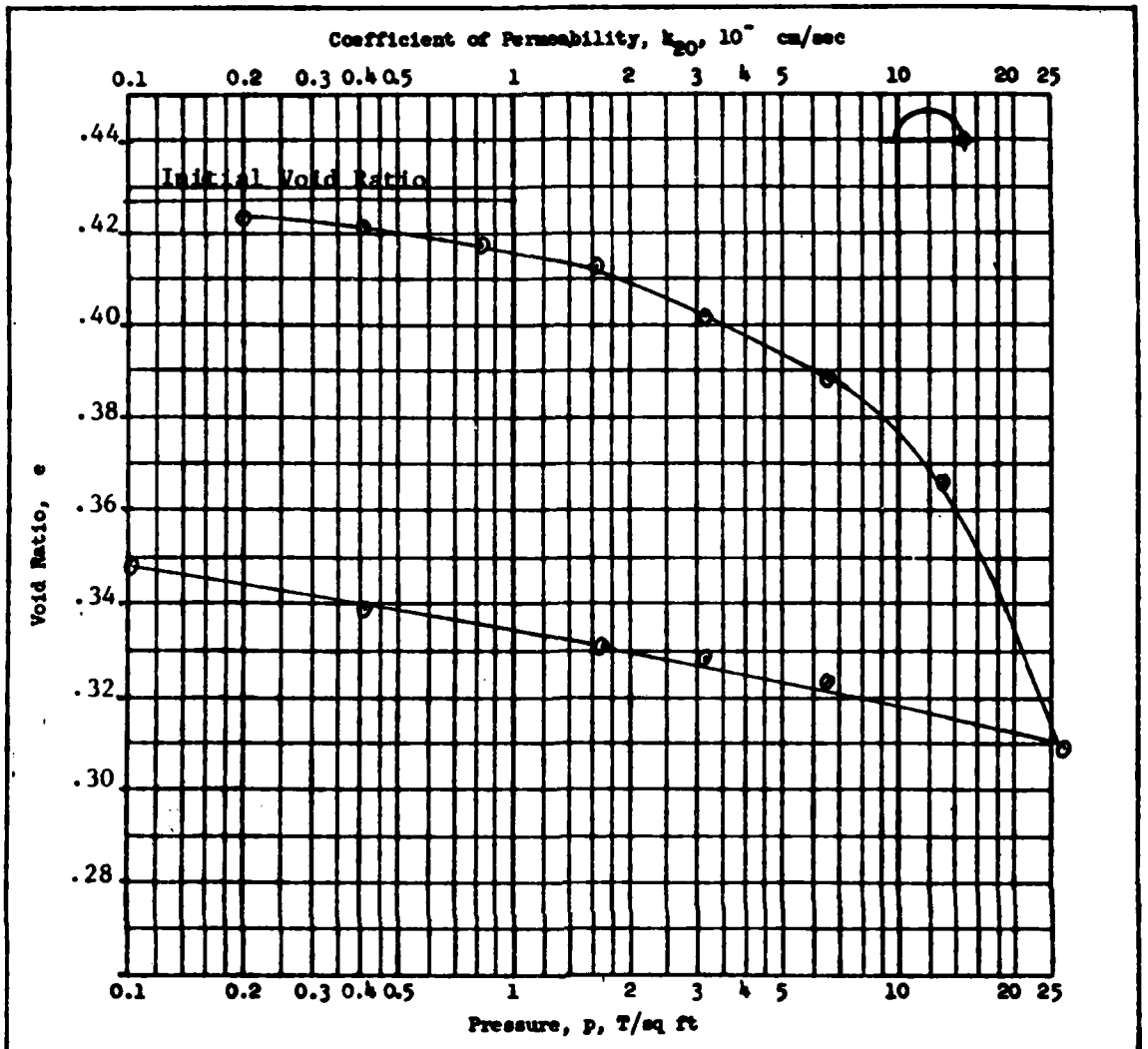
PLATE 17



PROJECT	Upper Santa Ana River			
AREA	Div. No. 96586 & 96587			
BORING NO.	TH-4 & 9	SAMPLE NO. Comp A	DEPTH EL 4-6	DATE August 1986

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST–TIME CURVES** (TRANSLUCENT)

PLATE 18

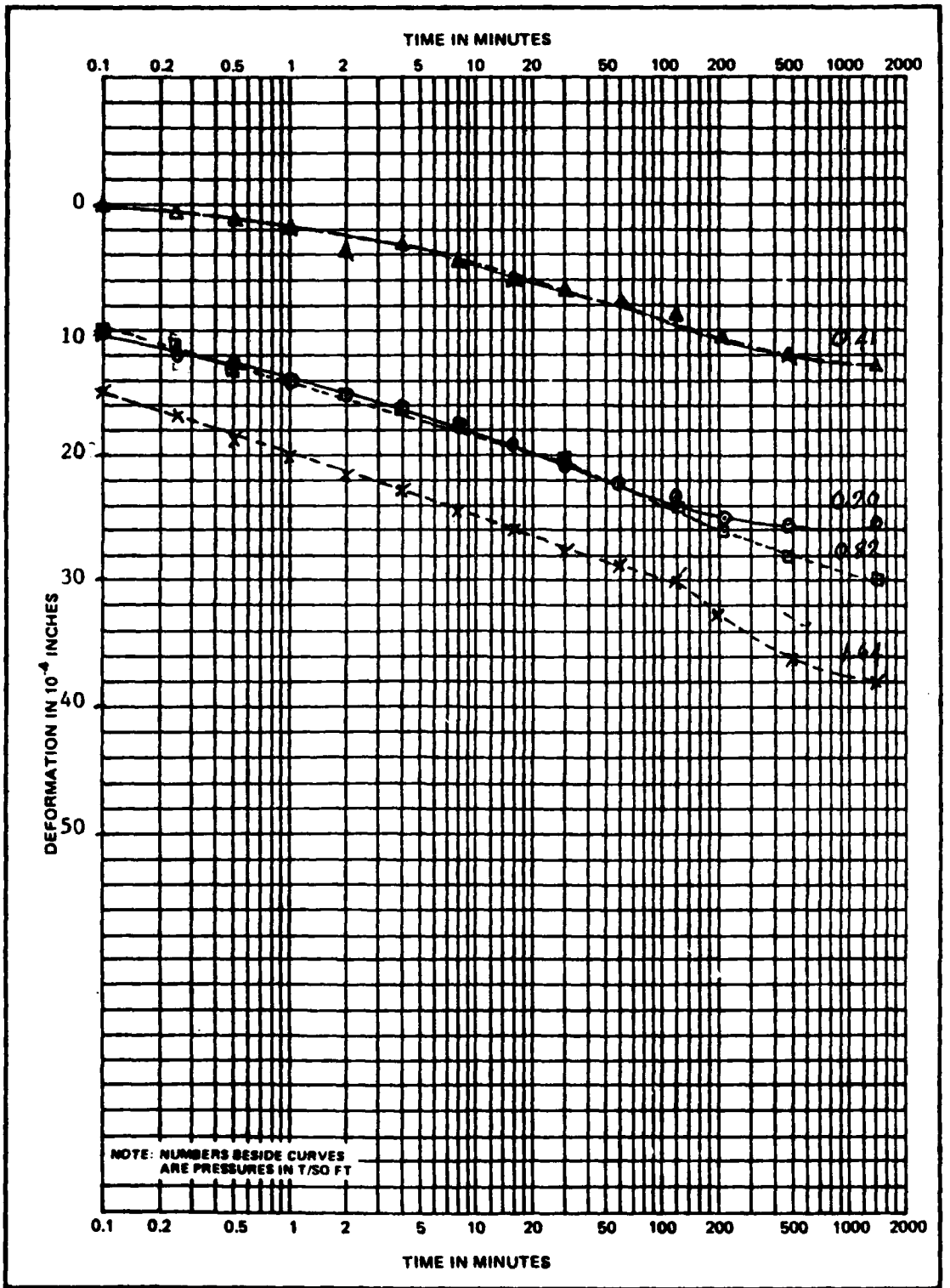


Type of Specimen: Remolded		Before Test		After Test	
Diam in.	Ht in.	Water Content, w_o	12.0 %	w_f	13.2 %
Overburden Pressure, Po T/sq ft		Void Ratio, e_o	0.427	e_f	0.358
Preconsol. Pressure, Pc T/sq ft		Saturation, S_o	75 %	S_f	99 %
Compression Index, C_c		Dry Density, γ_d	117.7 lb/ft^3		
Classification: Clayey Sand(SC)		k_{20} at e_o =	x 10^{-} cm/sec		
LL 27	G_s 2.69	Project: Upper Santa Ana River			
PL 18	D_{10}				
Remarks: Remolded to 90% max. density at 3% above optimum water content.		Area: Div. No. 97586 - 97587			
		Boring No. TH-85 4 & 9		Sample No. Blend "A"	
		Depth El 4' and 6'		Date: October 1986	
Div. No. 97586 - 96587		CONSOLIDATION TEST REPORT			

ENG FORM 1 MAY 68 2090 PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT)

GPO 926-736

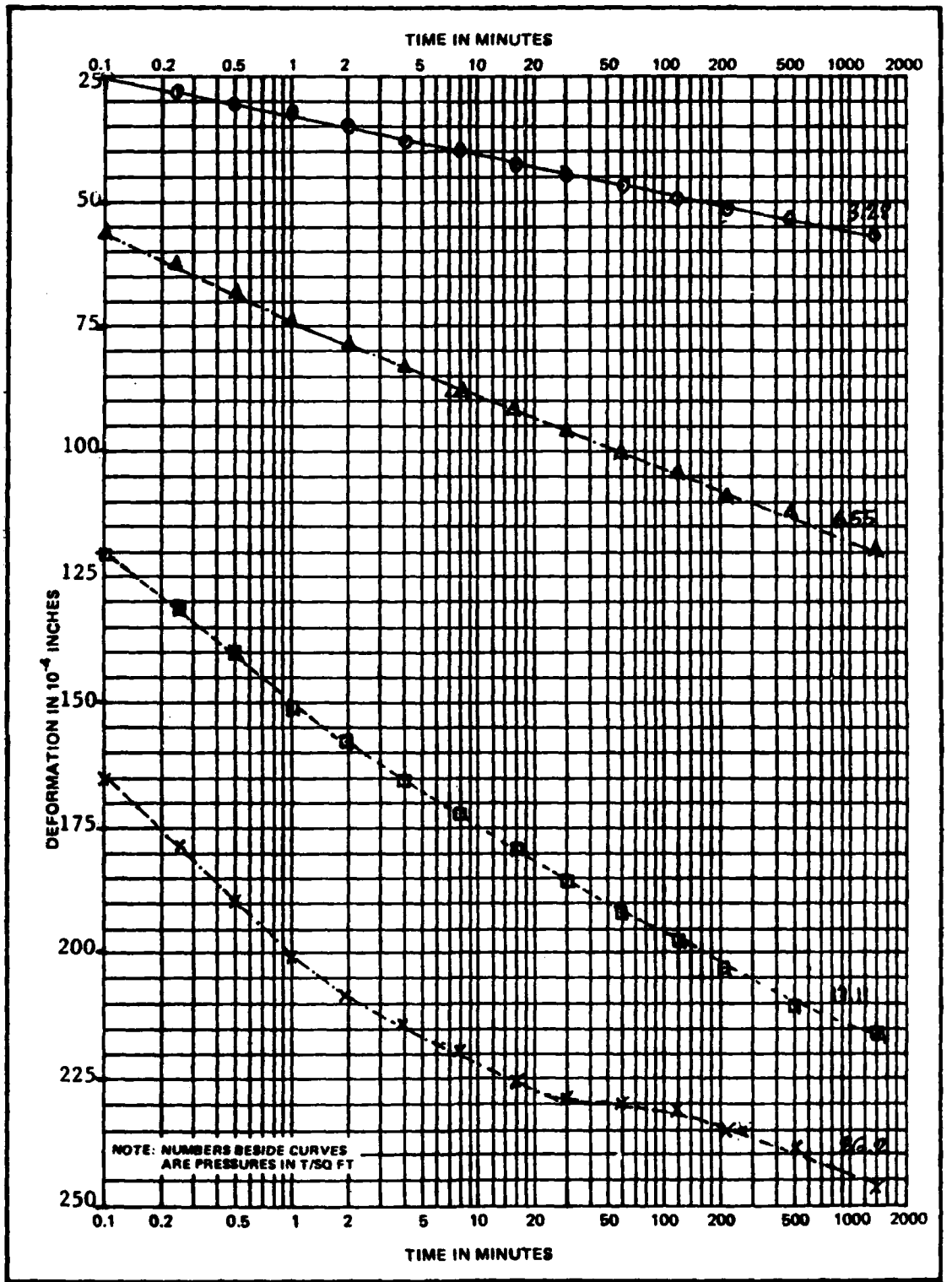
PLATE 13



PROJECT	Upper Santa Ana River			
AREA	Div. No. 97586 - 97587			
BORING NO.	TH-85-4 TH-85-9	SAMPLE NO. Blend "A"	DEPTH EL 4' and 6'	DATE October 1986

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST–TIME CURVES** (TRANSLUCENT)

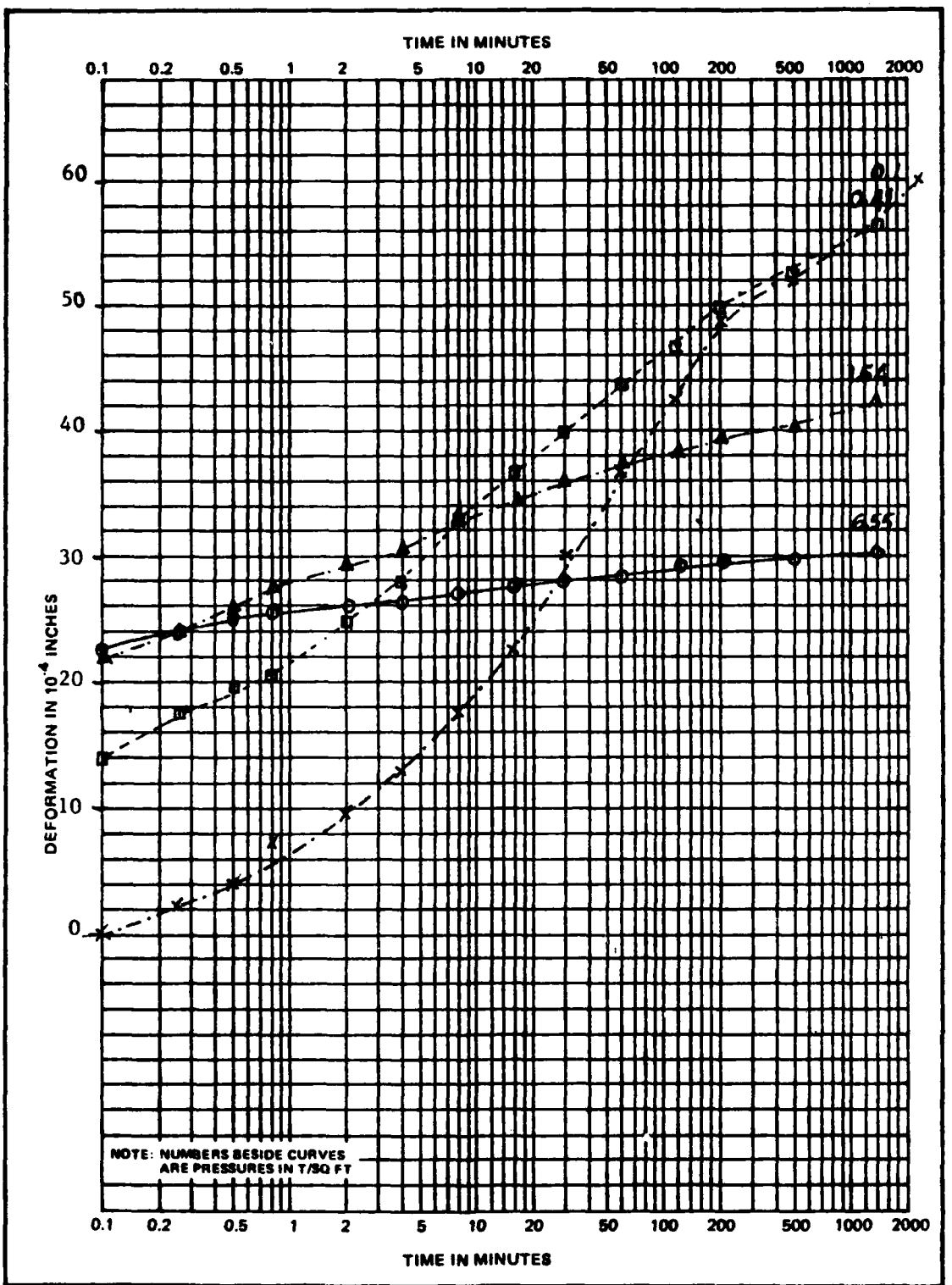
PLATE 20



PROJECT	Upper Santa Ana River			
AREA	Div. No. 97586 - 97587			
BORING NO.	TH-85-6 TH-85-9	SAMPLE NO. Blend "A"	DEPTH EL 4' and 6'	DATE October 1986

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST–TIME CURVES** (TRANSLUCENT)

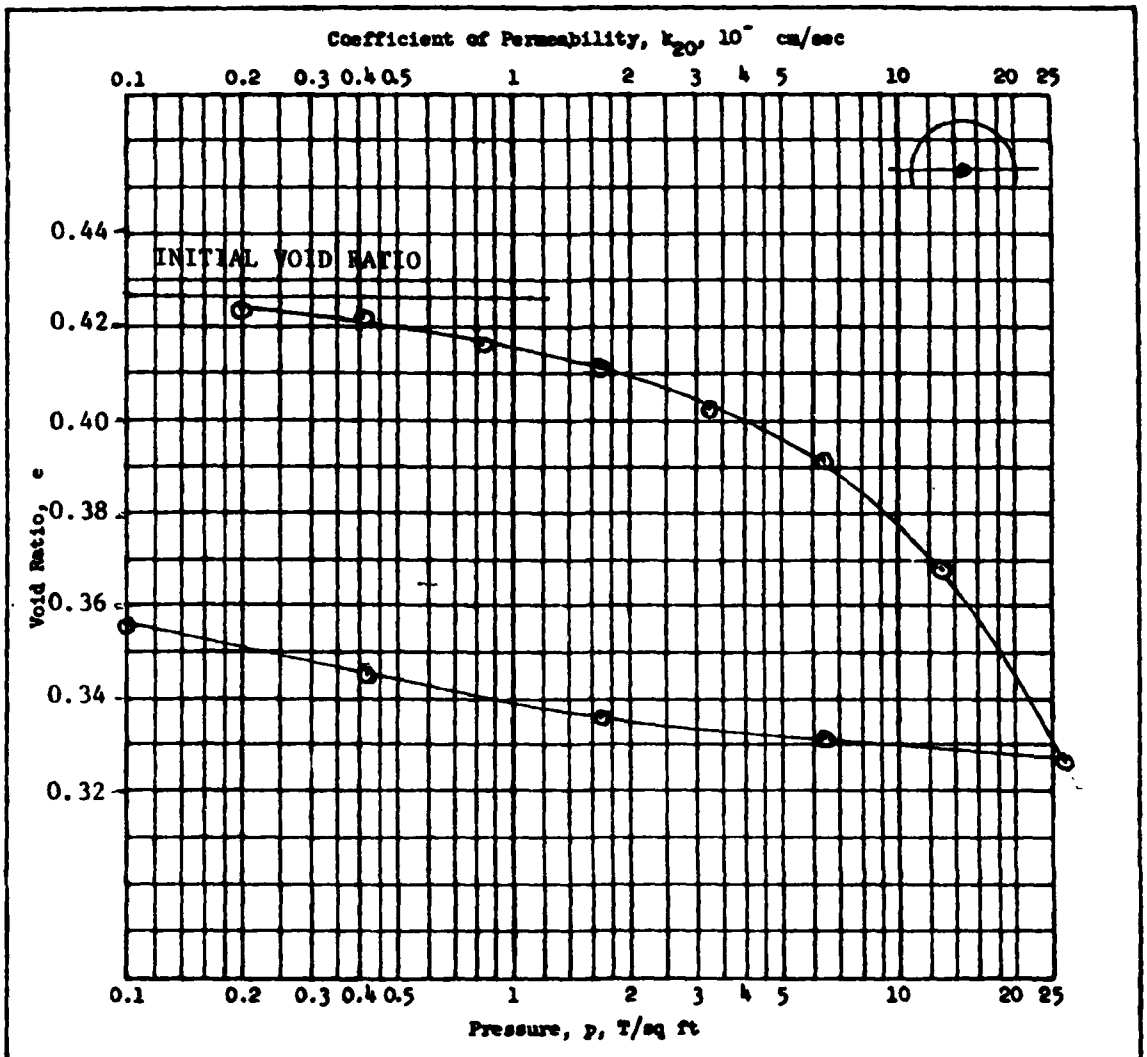
PLATE 21



PROJECT	Upper Santa Ana River			
AREA	Div. No. 97586 - 96587			
BORING NO.	TH-85-6 TH-85-9	SAMPLE NO Blend "A"	DEPTH EL 4'and 6'	DATE October 1986

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST–TIME CURVES** (TRANSLUCENT)

PLATE 22

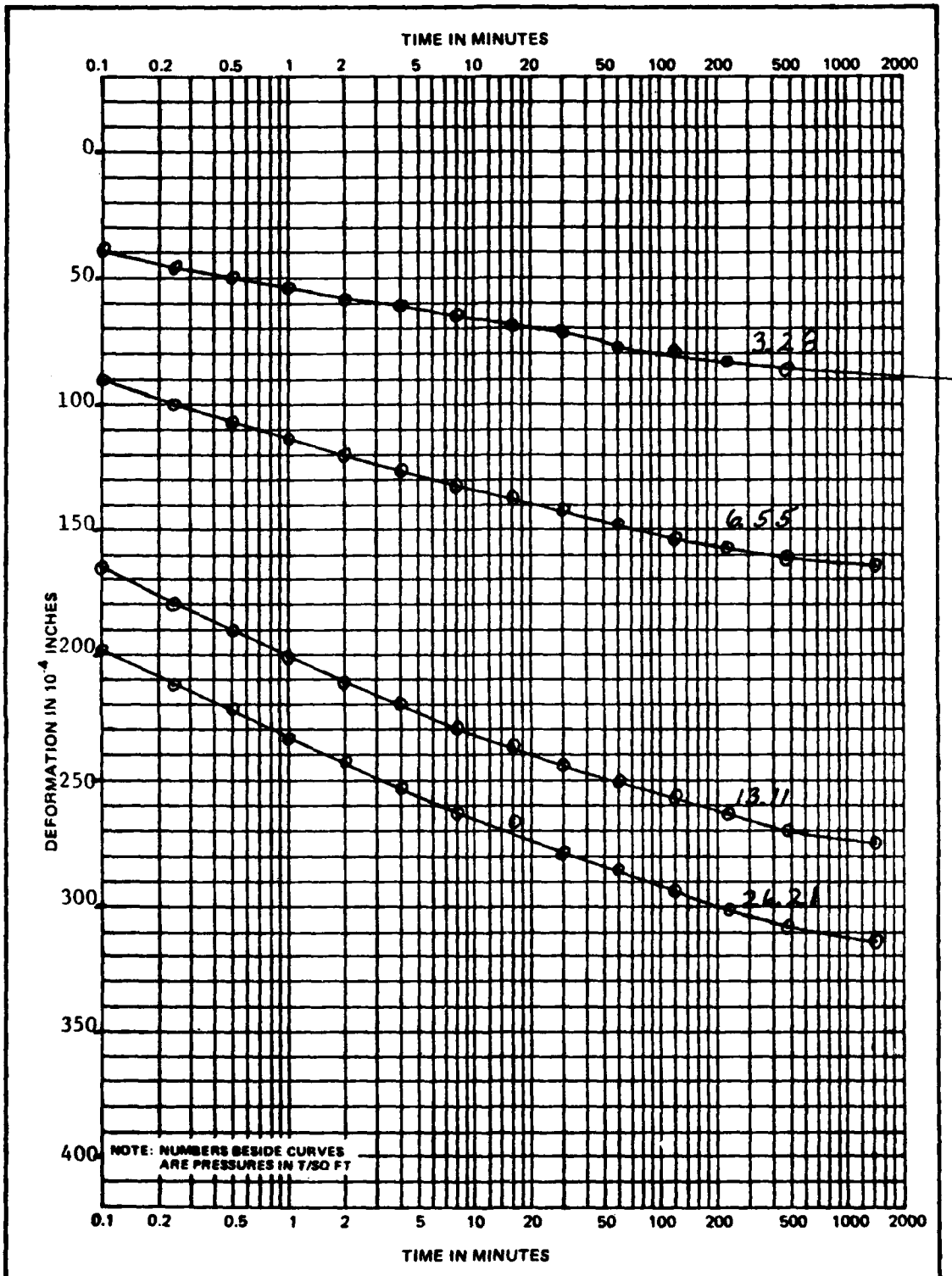


Type of Specimen: Remolded		Before Test		After Test	
Diam 4.4 in.	Ht 1.00 in.	Water Content, w_o	8.9 %	w_f	12.9 %
Overburden Pressure, P_o T/sq ft		Void Ratio, e_o	0.427	e_f	0.368
Preconsol. Pressure, P_c T/sq ft		Saturation, S_o	56 %	S_f	95 %
Compression Index, C_c		Dry Density, γ_d	117.7 lb/ft^3		
Classification Clayey Sand(SC)		k_{20} at e_o =	$\times 10^{-}$ cm/sec		
LL 27	G_s 2.69	Project Upper Santa Ana River			
PL 18	D_{10}				
Remarks Remolded to 90%		Area Impervious Borrow			
Max. density at optimum water		Boring No. TH-85-4		Sample No. Comp. A	
content.		Depth El 4-6		Date August 1986	
Div. No. 96586 - 96587		CONSOLIDATION TEST REPORT			

ENG FORM 1 MAY 63 2090 PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT)

GPO 936-736

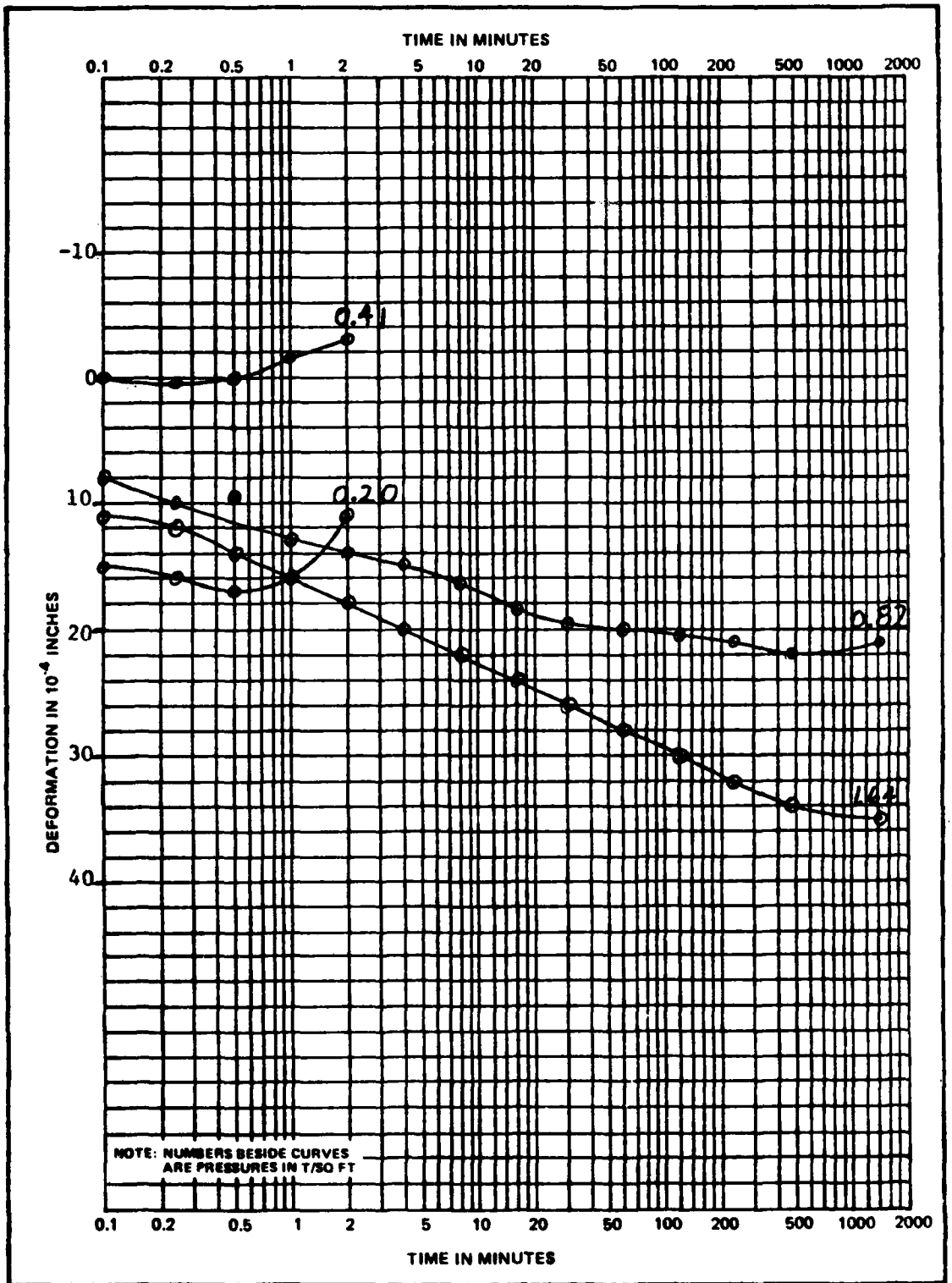
0 3424 PLATE 23



PROJECT Upper Santa Ana River Dam Div. No. 96586-96587			
AREA Impervious Core Material, Blend "A"			
BORING NO TH-85-4 &9	SAMPLE NO.	DEPTH EL 4' & 6'	DATE August 1986

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST–TIME CURVES** (TRANSLUCENT)

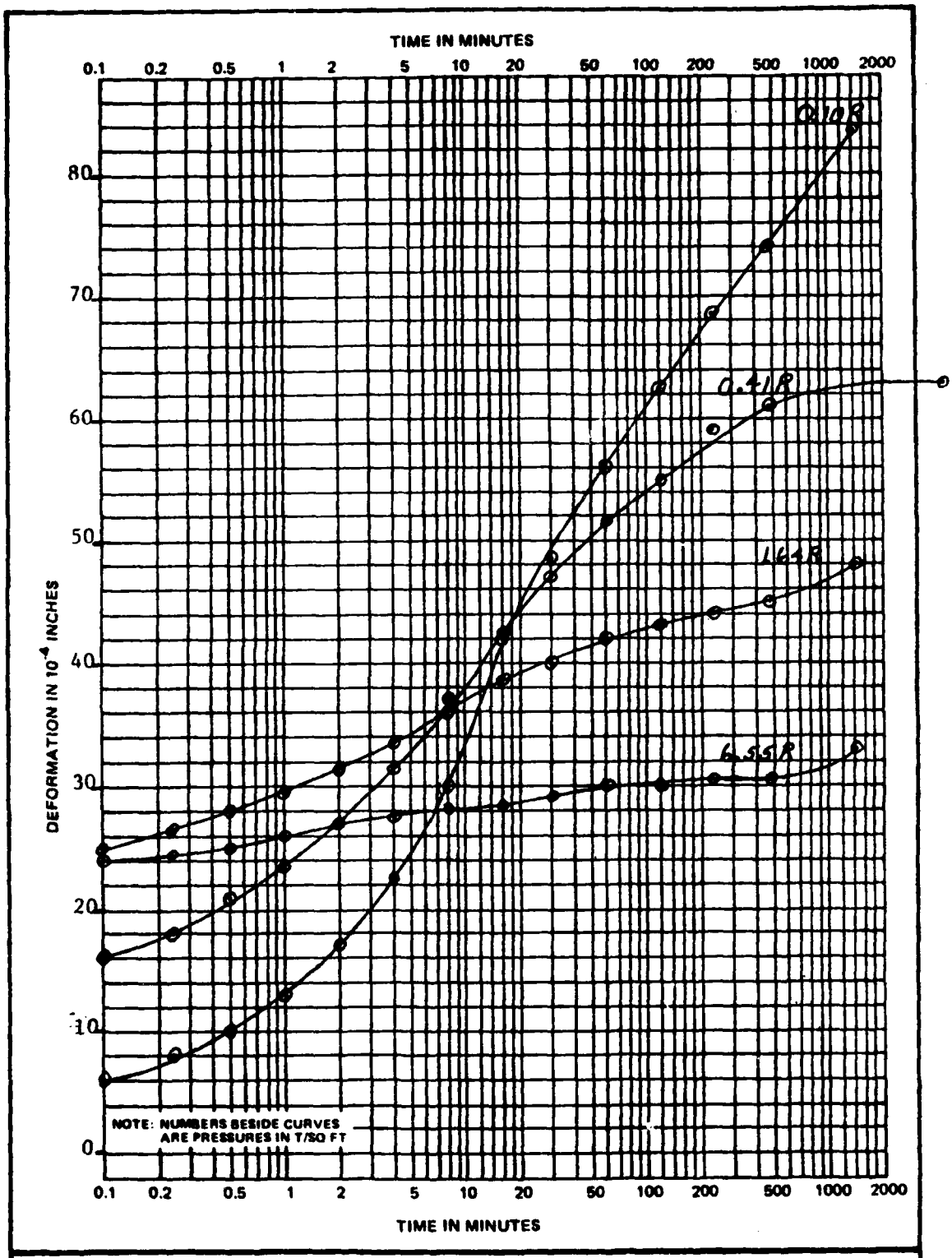
PLATE 24



PROJECT Upper Santa Ana River Dam Div. No. 96586-96587			
AREA Impervious Core Material, Blend "A"			
BORING NO. TH-85 4 & 9	SAMPLE NO.	DEPTH EL 4' & 6'	DATE Aug. 1986

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST–TIME CURVES** (TRANSLUCENT)

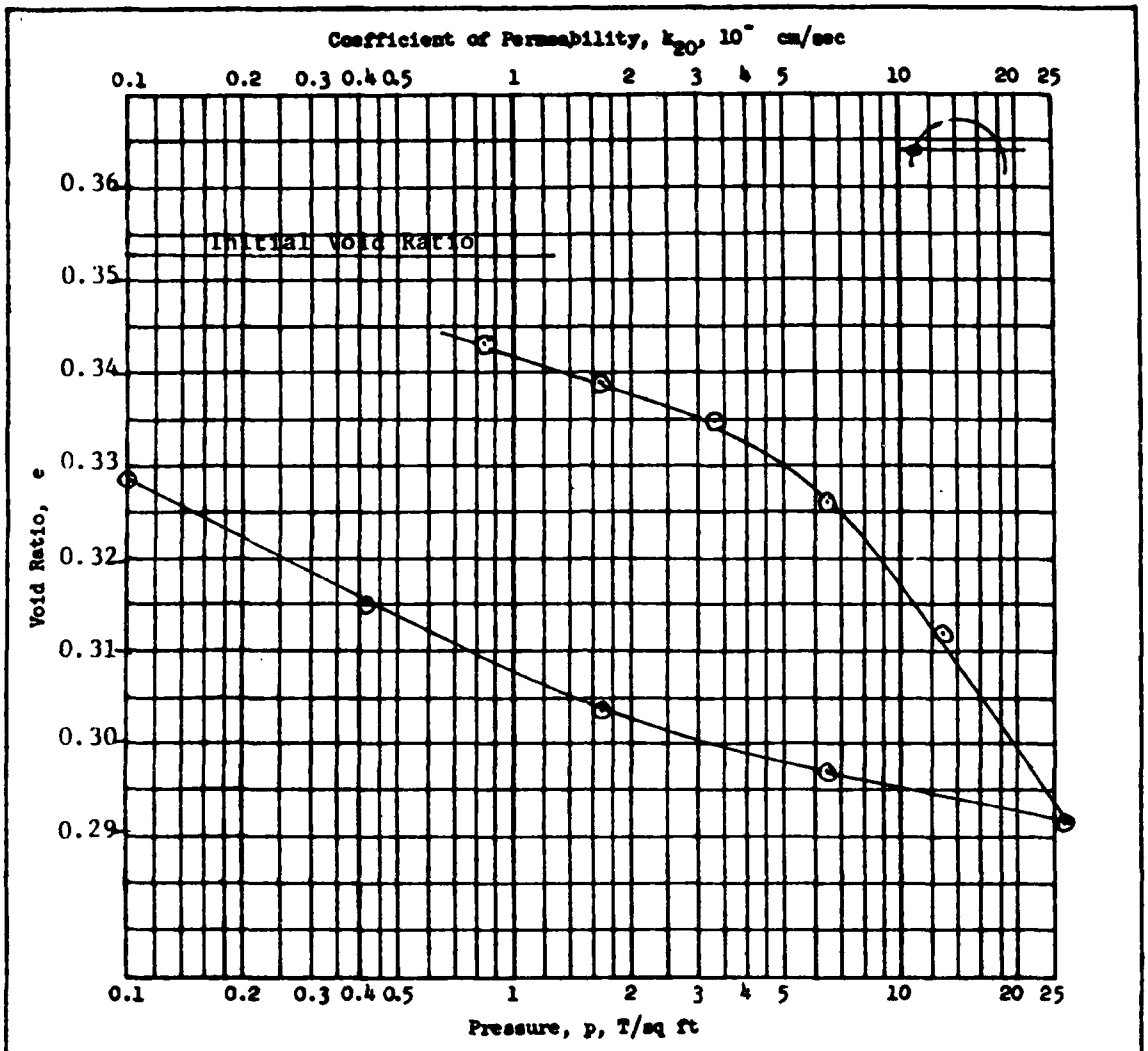
PLATE 25



PROJECT	Upper Santa Ana River Dam Div. No. 96586-96587			
AREA	Impervious Core Material, Blend "A"			
BORING NO. TH-85 4&9	SAMPLE NO.	DEPTH EL	4' & 6'	DATE August 1986

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST–TIME CURVES** (TRANSLUCENT)

PLATE 26

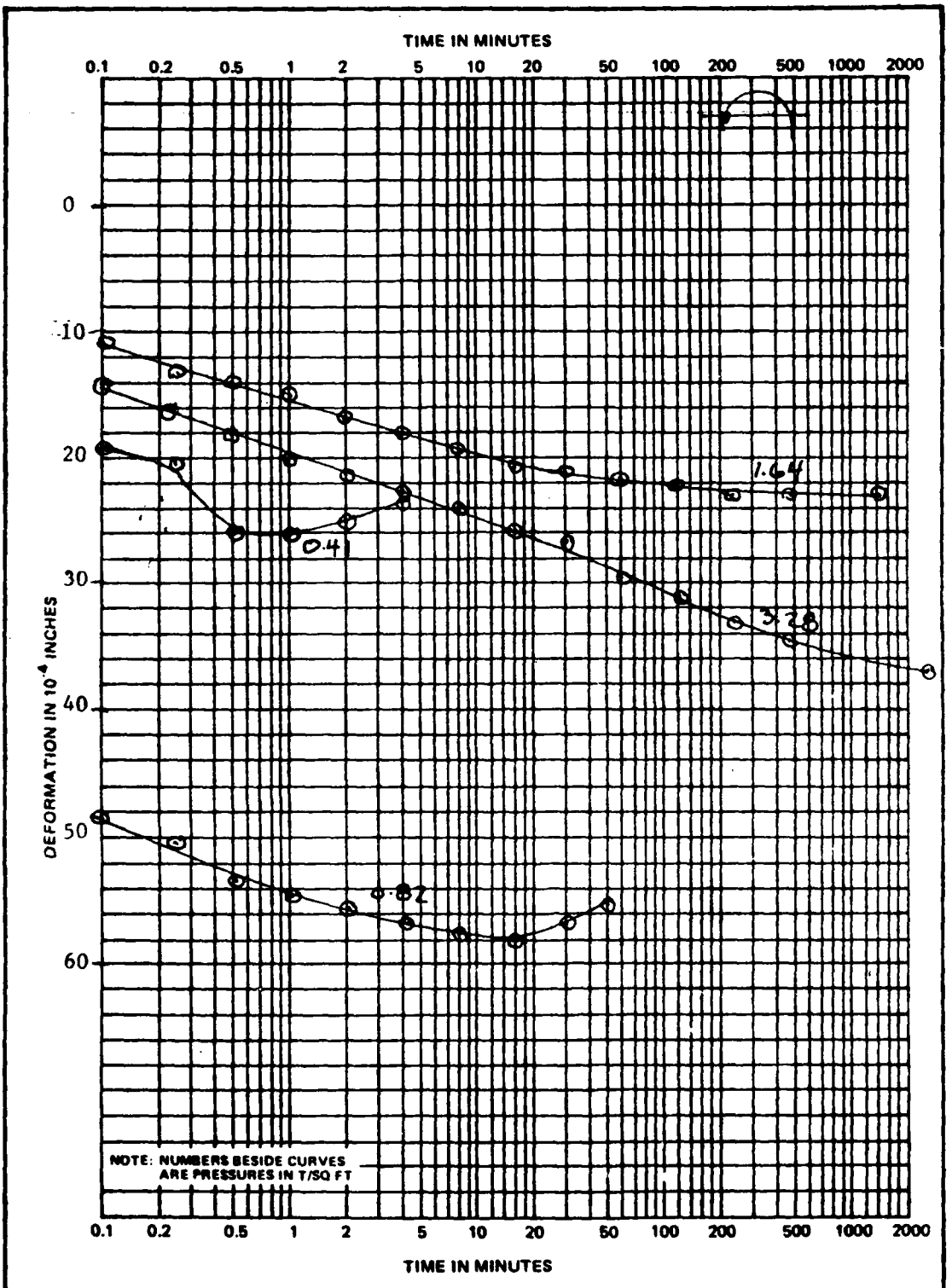


Type of Specimen Remolded		Before Test		After Test	
Diam 4.4 in.	Ht 1.00 in.	Water Content, w_o	5.9 %	w_f	13.8 %
Overburden Pressure, p_o T/sq ft		Void Ratio, e_o	0.353	e_f	0.390
Preconsol. Pressure, p_c T/sq ft		Saturation, S_o	45 %	S_f	95 %
Compression Index, C_c		Dry Density, γ_d	124.0 lb/ft^3		
Classification Clayey Sand (SC)		k_{20} at e_o =	x 10^{-} cm/sec		
LL 27	G_s 2.69	Project Upper Santa Ana River			
PL 18	D_{10}				
Remarks Composite: Div. No.		Area Impervious Borrow			
96586 & 96587		Boring No. TH-85-4&9		Sample No. Comp. A	
Remolded to 95% max. at 3%		Depth El 4-6		Date August 1986	
below optimum w.c.		CONSOLIDATION TEST REPORT			

ENG FORM 1 MAY 69 2090 PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT)

GPO 936-736

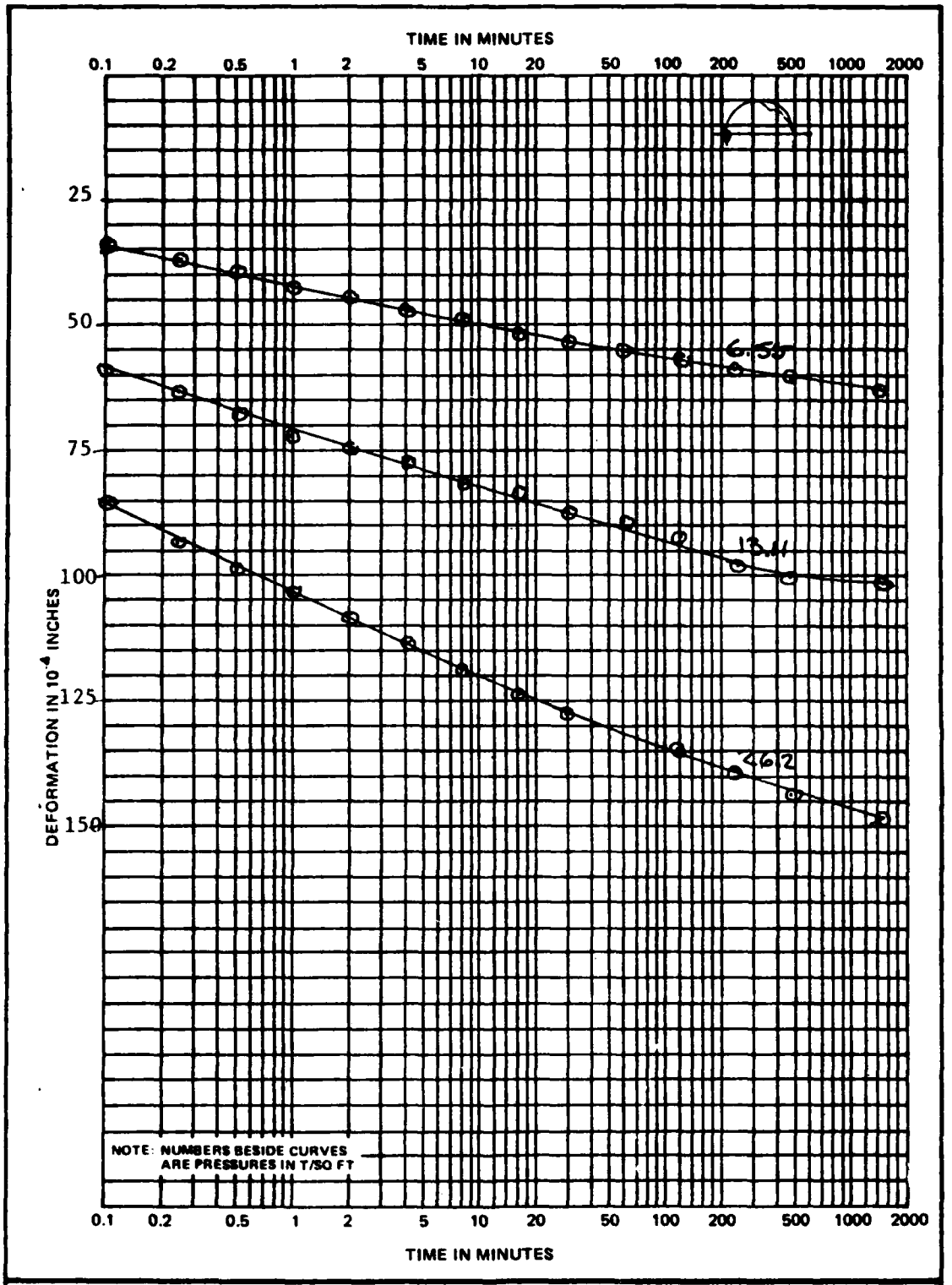
n 3424 PLATE 27



PROJECT	Upper Santa Ana River, Div. No. 96586 - 96587		
AREA	Impervious Borrow		
BORING NO. TH-85-4&9	SAMPLE NO. Comp A	DEPTH EL 4-6	DATE August 1986

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST--TIME CURVES** (TRANSLUCENT)

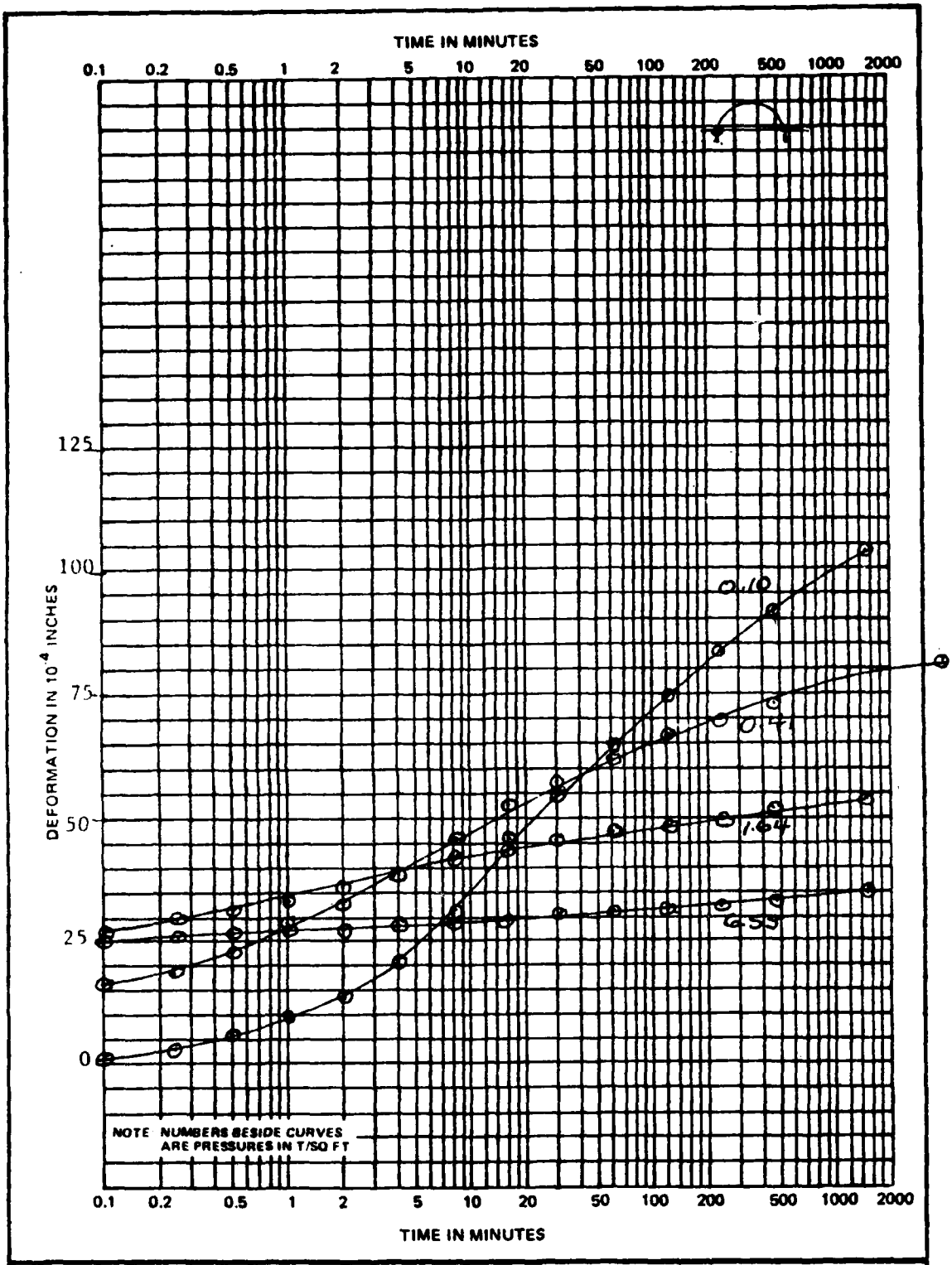
PLATE 28



PROJECT	Upper Santa Ana River , Div. No. 96586 - 96587			
AREA	Impervious Borrow			
BORING NO.	TH-85-4 &9	SAMPLE NO. Comp A	DEPTH EL 4-6	DATE August 1986

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST--TIME CURVES** (TRANSLUCENT)

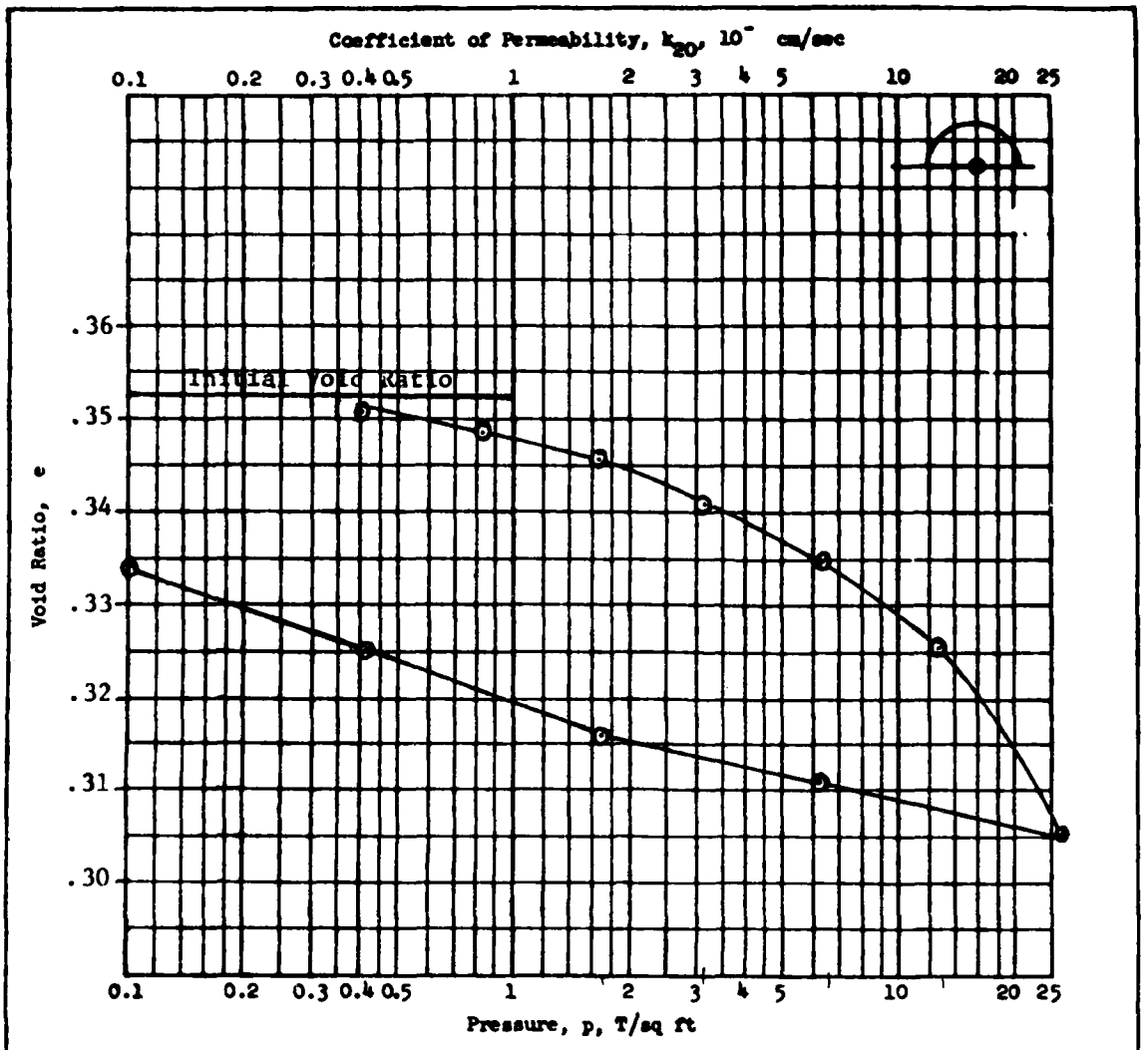
PLATE 29



PROJECT	Upper Santa Ana River , Div. No. 96586 - 96587			
AREA	Impervious Borrow			
BORING NO.	TH-85-4 & 9	SAMPLE NO Comp A	DEPTH EL 4-6	DATE August 1986

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST--TIME CURVES** (TRANSLUCENT)

PLATE 30

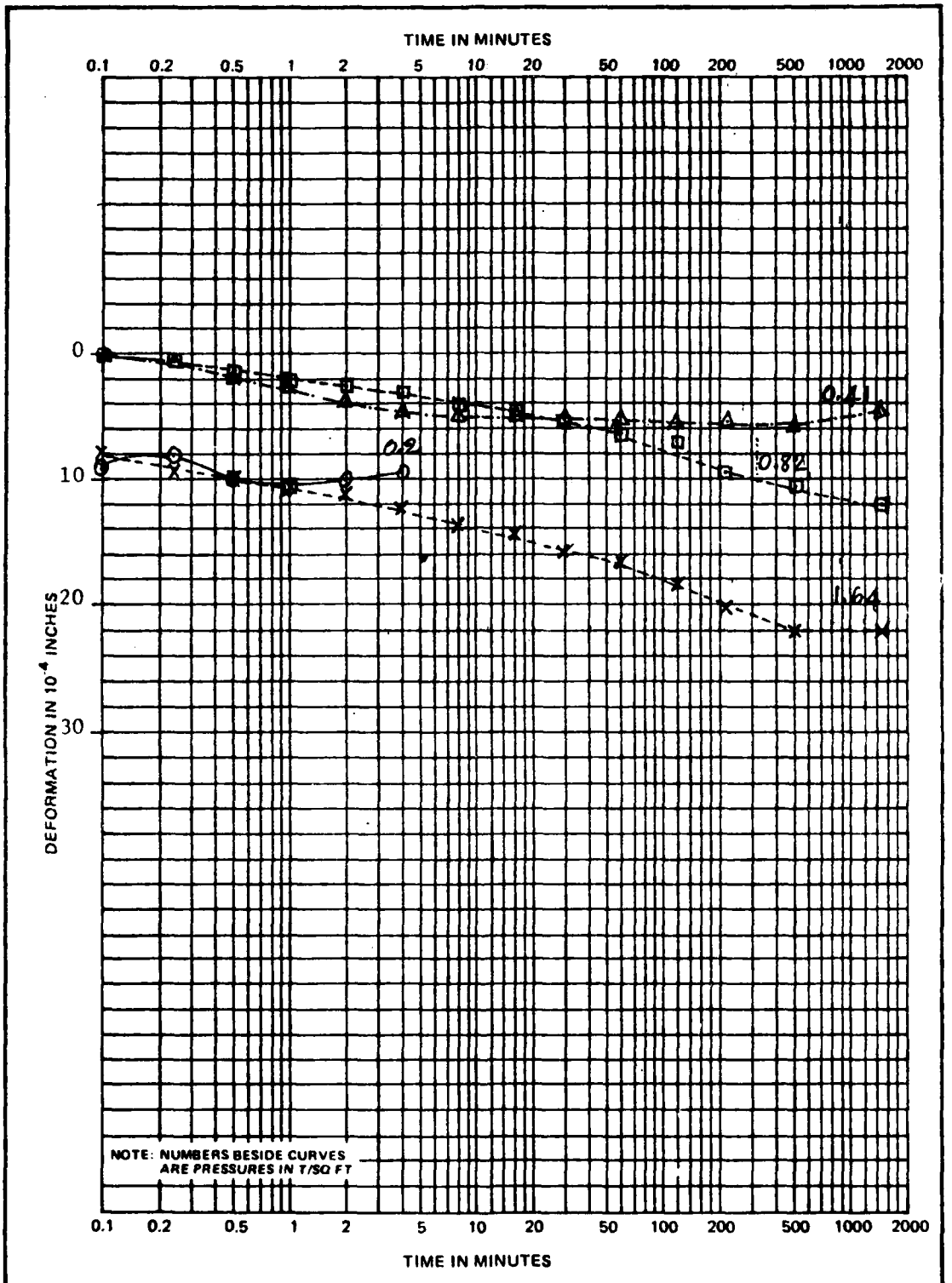


Type of Specimen Remolded		Before Test		After Test	
Diam in.	Ht in.	Water Content, w_o	8.9 %	w_f	12.6 %
Overburden Pressure, Po T/sq ft		Void Ratio, e_o	0.352	e_f	0.352
Preconsol. Pressure, Pc T/sq ft		Saturation, S_o	68.0 %	S_f	96.5 %
Compression Index, C_c		Dry Density, γ_d	124.1 lb/ft^3		
Classification Clayey Sand(SC)		k_{20} at e_o =	x 10^{-} cm/sec		
LL 27	G_s 2.69	Project Upper Santa Ana River			
PL 18	D_{10}				
Remarks Remolded to 95%		Area Impervious Borrow			
Div. No. 96586 - 96587		Boring No. TH-85-4		Sample No. Comp A	
		Depth El 4-6		Date Sept. 1986	
		CONSOLIDATION TEST REPORT			

ENG FORM 1 MAY 69 2090 PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT)

GPO 936-736

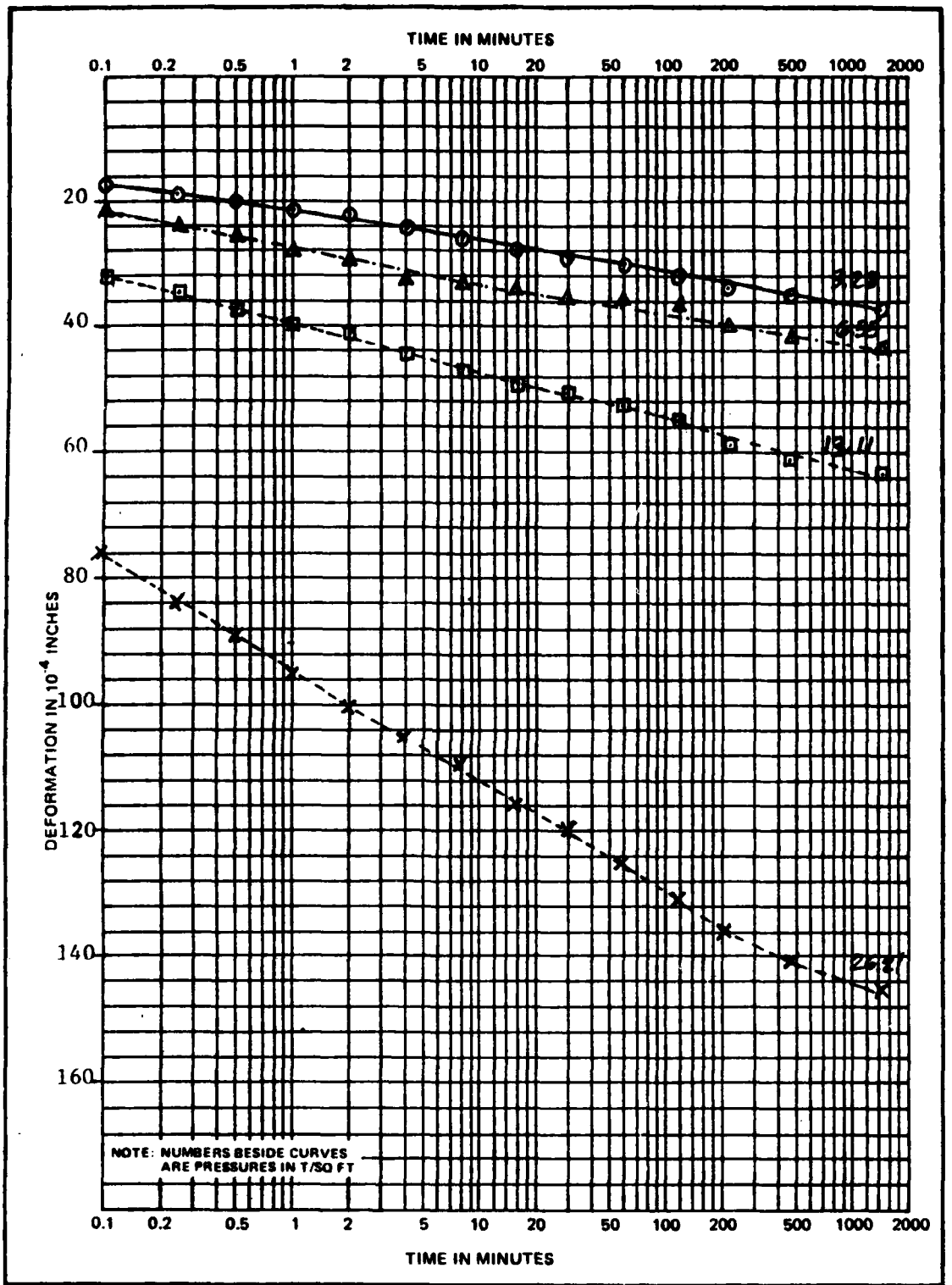
n 3424 PLATE 31



PROJECT Upper Santa Ana River			
AREA Div. No. 96586 & 96587			
BORING NO. TH-85-4&9	SAMPLE NO. Comp A	DEPTH EL 4-6	DATE Sept. 1986

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST--TIME CURVES** (TRANSLUCENT)

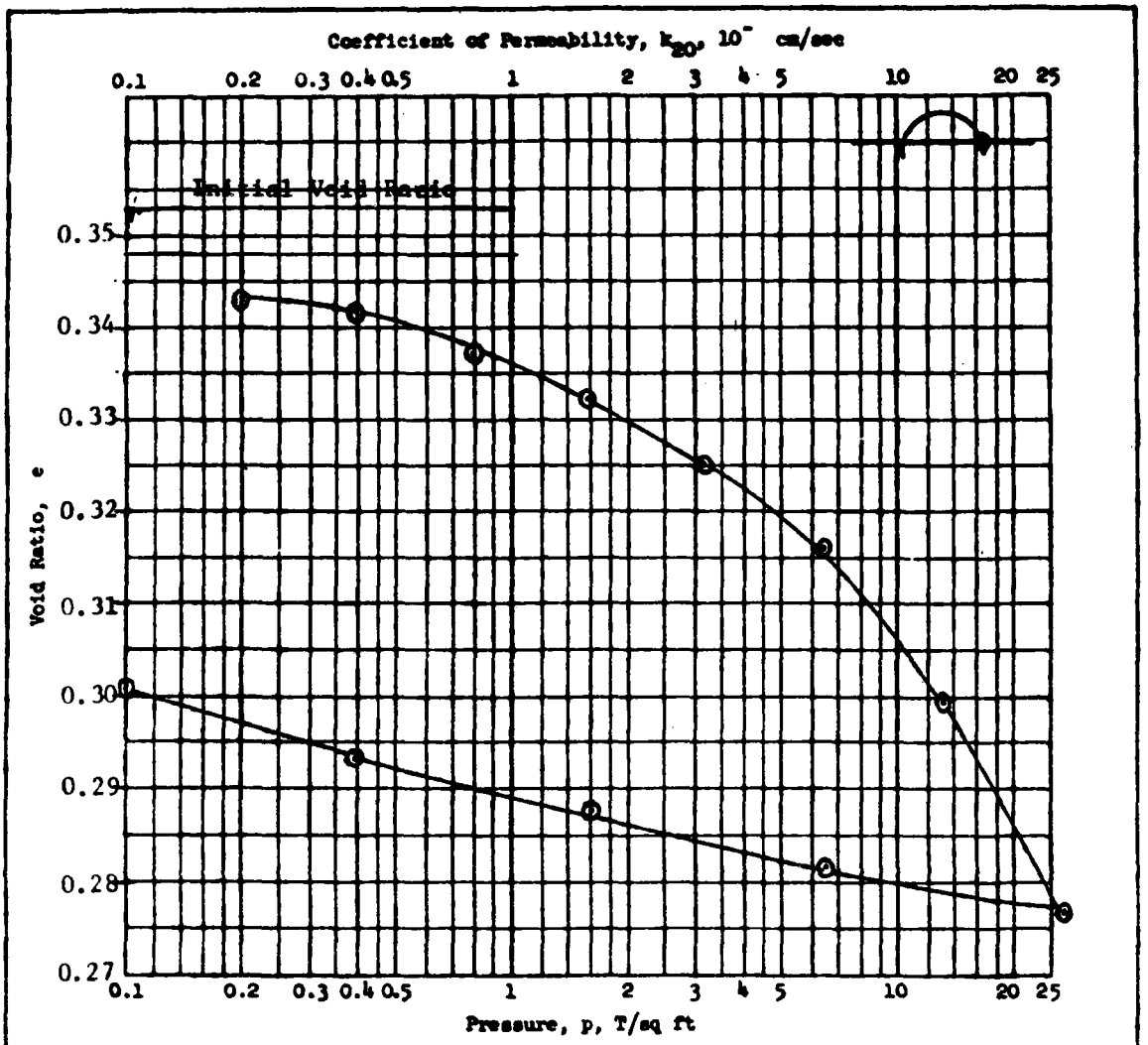
PLATE 32



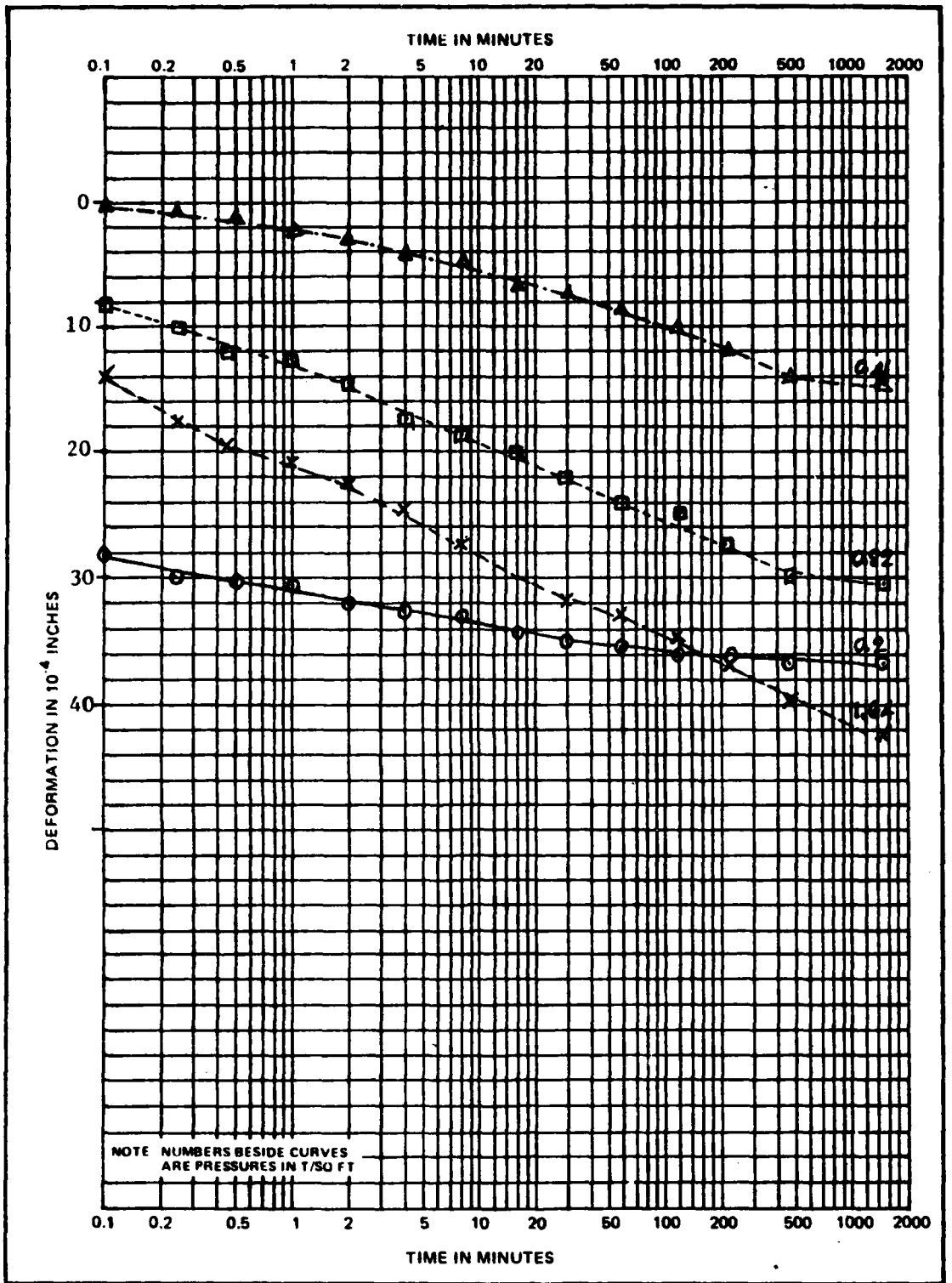
PROJECT	Upper Santa Ana River			
AREA	Div. No. 96586 & 96587			
BORING NO. TH-85 4 &9	SAMPLE NO. Comp A	DEPTH EL 4-6	DATE Sept. 1986	

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST–TIME CURVES** (TRANSLUCENT)

PLATE 33



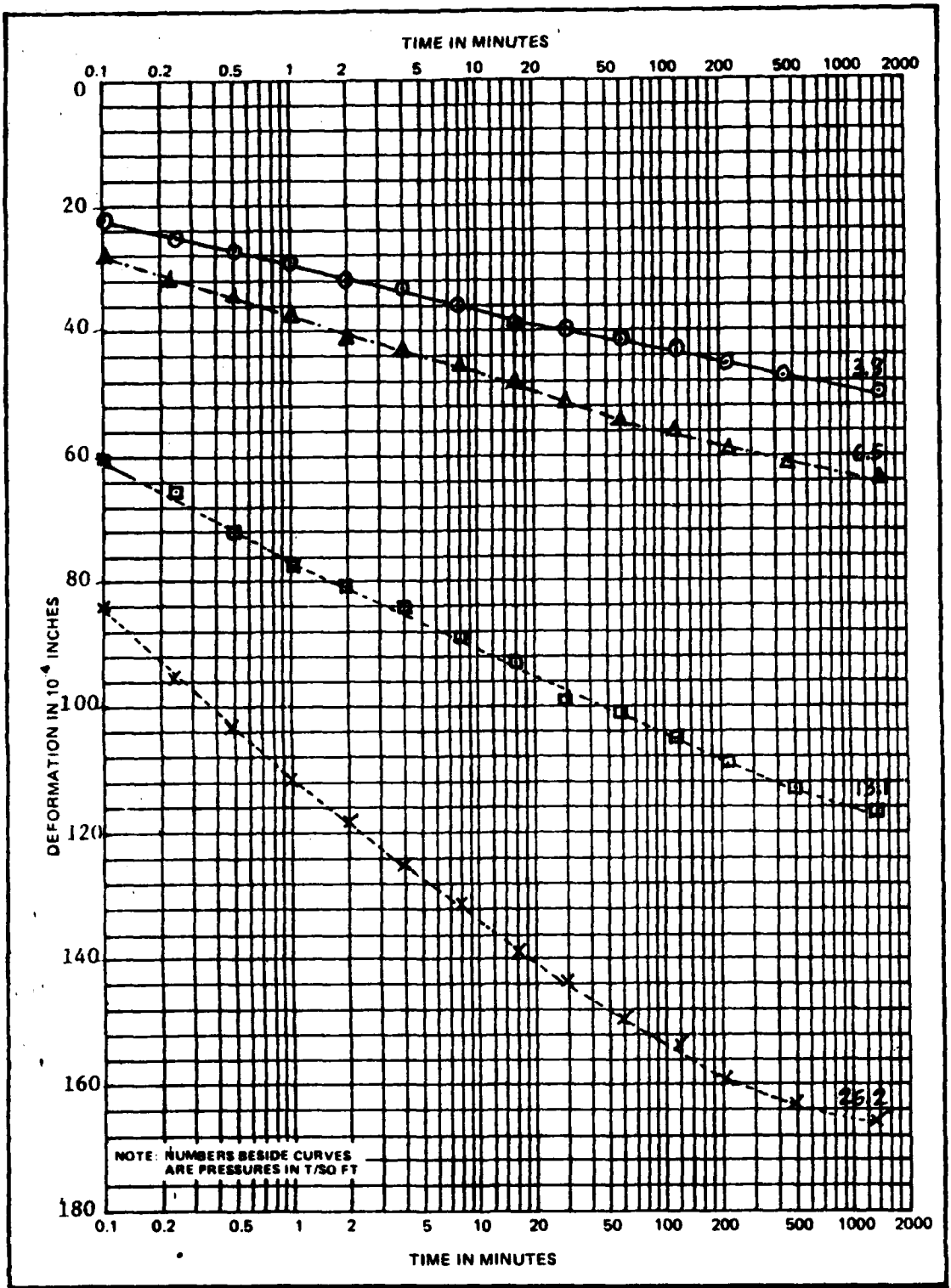
Type of Specimen Remolded		Before Test		After Test	
Diam 4.4 in.	Ht 1.00 in.	Water Content, w_o	11.3 %	w_f	11.8 %
Overburden Pressure, p_o T/sq ft		Void Ratio, e_o	0.348	e_f	0.317
Preconsol. Pressure, p_c T/sq ft		Saturation, S_o	87.5 %	S_f	100 %
Compression Index, C_c		Dry Density, γ_d	124.5 lb/ft^3		
Classification Clayey Sand (SC)		k_{20} at e_o =	x 10^{-} cm/sec		
LL 27	G_s 2.69	Project Upper Santa Ana			
PL 18	D_{10}				
Remarks Remolded to 95% max. density at 2.5% above optimum water content.		Area Div. No. 96586 & 96587			
		Boring No. TH-85-4 & 9	Sample No. Comp A		
		Depth El 4-6	Date Sept. 1986		
		CONSOLIDATION TEST REPORT			



PROJECT	Upper Santa Ana			
AREA	Div. No. 96586 & 96587			
BORING NO.	TH-85-4 & 9	SAMPLE NO. Comp A	DEPTH EL 4-6	DATE Sept. 1986

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST–TIME CURVES** (TRANSLUCENT)

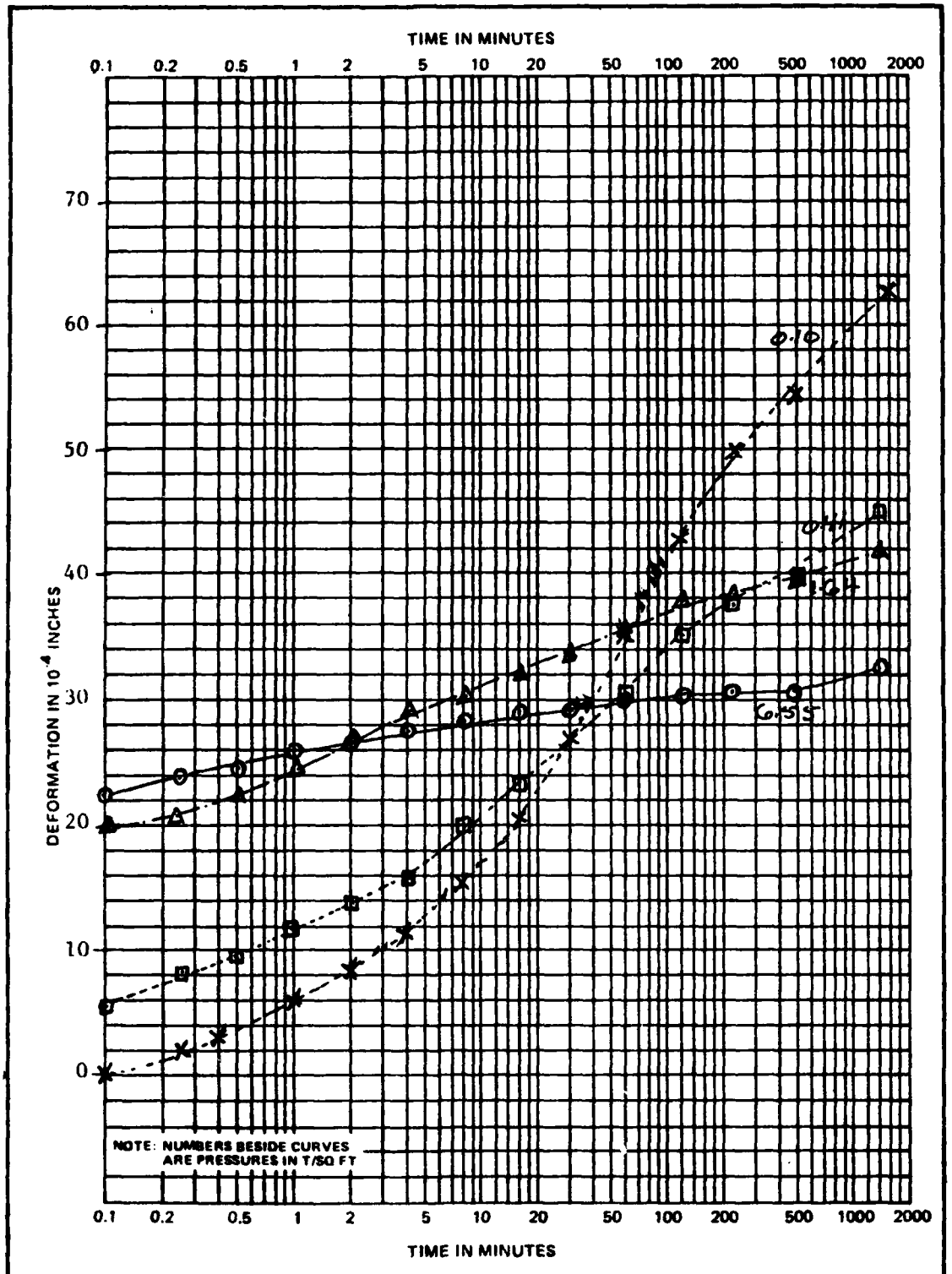
PLATE 35



PROJECT	Upper Santa Ana		
AREA	Div. No. 96586 & 96587		
BORING NO. TH-85 4 & 9	SAMPLE NO Comp A	DEPTH EL 4-6	DATE Sept. 1986

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST–TIME CURVES** (TRANSLUCENT)

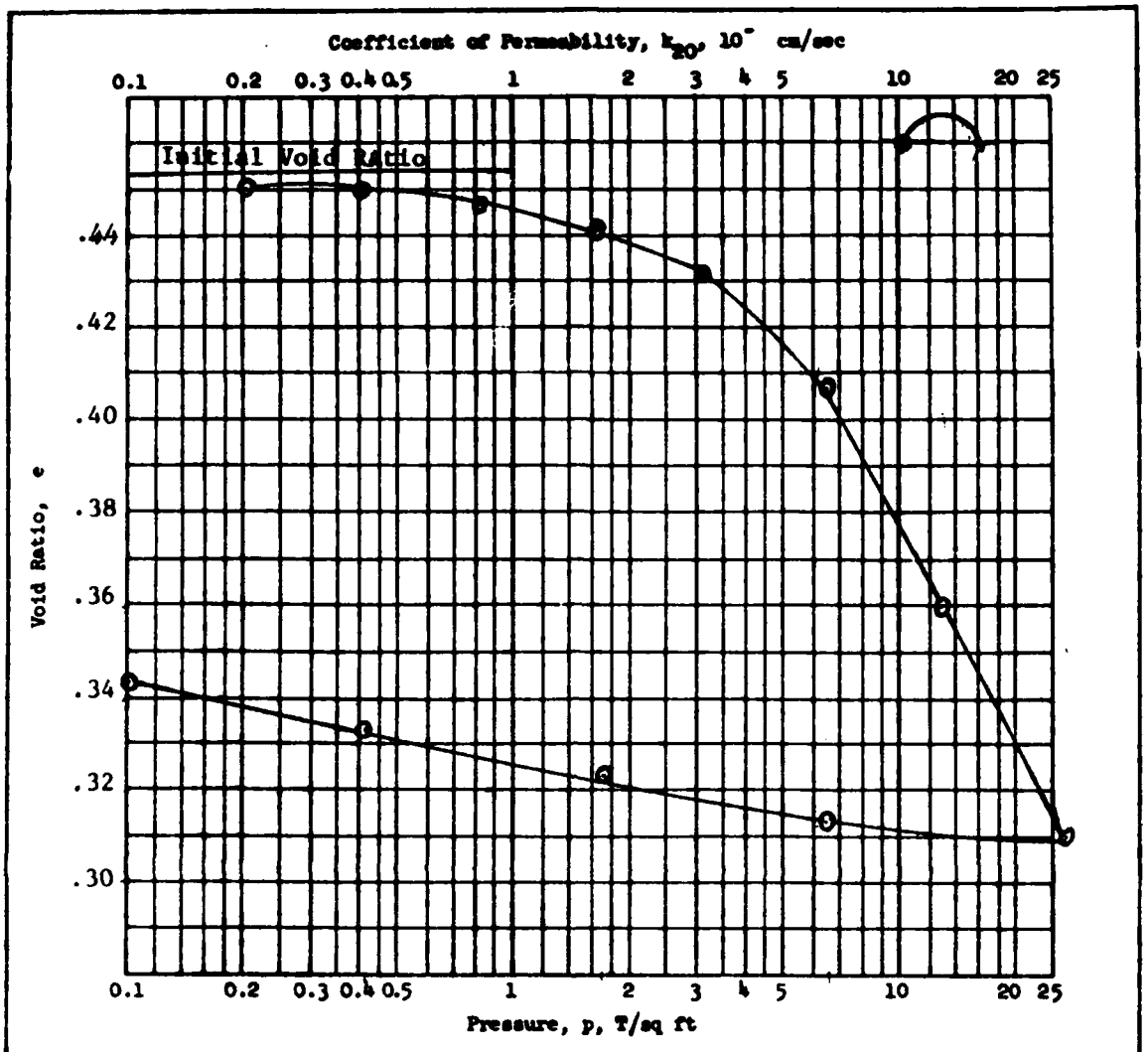
PLATE 36



PROJECT	Upper Santa Ana		
AREA	Div. No. 96586 & 96587		
BORING NO. TH-85-4&9	SAMPLE NO. Comp A	DEPTH EL 4-6	DATE September 1986

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST—TIME CURVES** (TRANSLUCENT)

PLATE 37



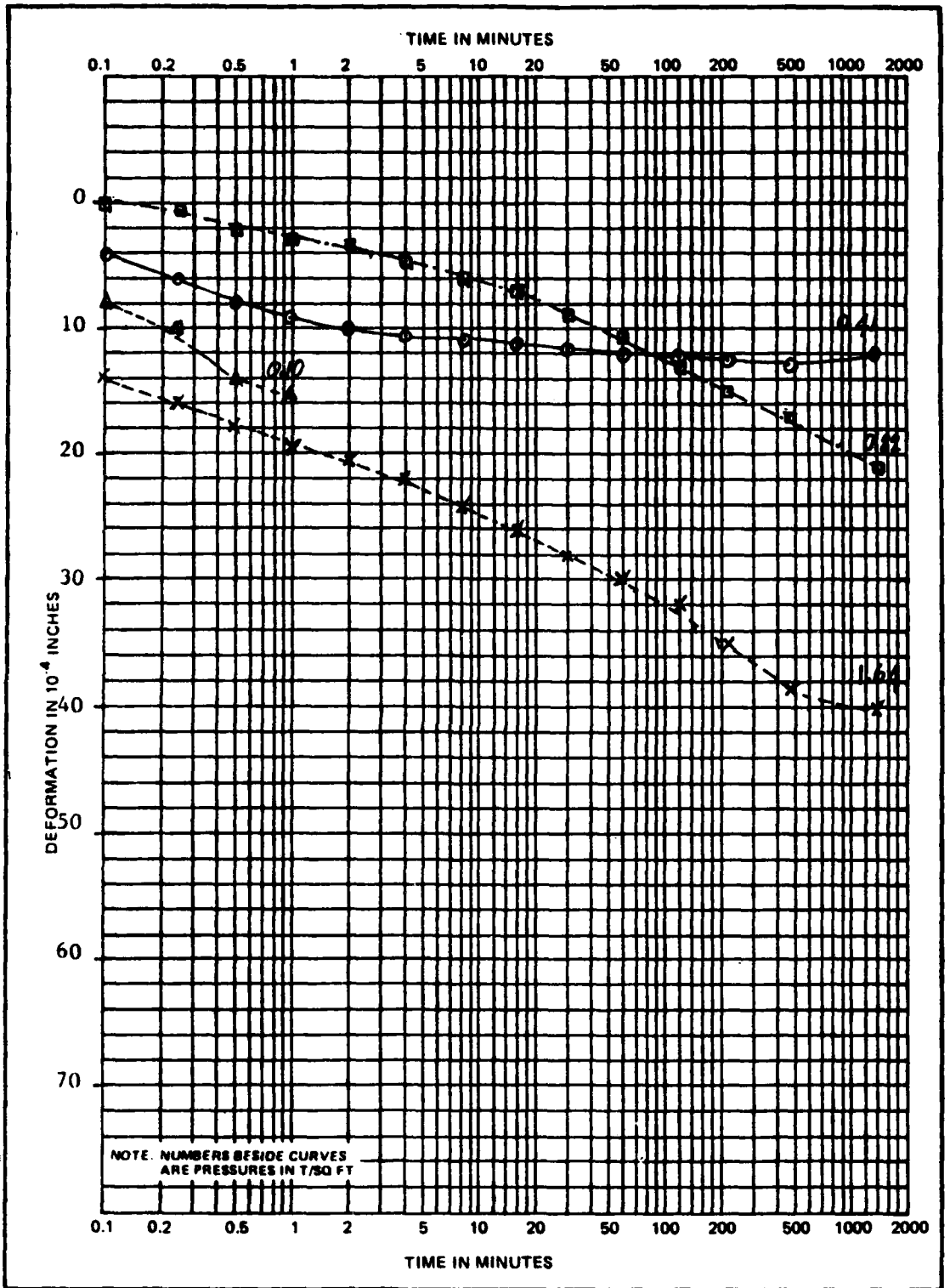
Type of Specimen Remolded		Before Test		After Test	
Diam in.	Ht in.	Water Content, w_o	9.4 %	w_f	13.4 %
Overburden Pressure, p_o T/sq ft		Void Ratio, e_o	0.453	e_f	0.362
Preconsol. Pressure, p_c T/sq ft		Saturation, S_o	57 %	S_f	100 %
Compression Index, C_c		Dry Density, γ_d	116.4 lb/ft^3		
Classification Silty Clayey Sand(SC) Clayey Sand(SC)		k_{20} at e_o =	x 10^{-} cm/sec		
LL 31 & 29	G_s 2.71	Project Upper Santa Ana River			
PL 18 & 21	D_{10}				
Remarks Remolded to 98% max. density at 3% below optimum water content.		Div. No. 97595 - 97596			
Div. No. 97595 - 97596		Boring No. TH-21, 23	Sample No. Blend "C"		
		Depth El 13' & 15'	Date October 1986		

CONSOLIDATION TEST REPORT

ENG FORM 1 MAY 63 2990 PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT)

GPO 936-736

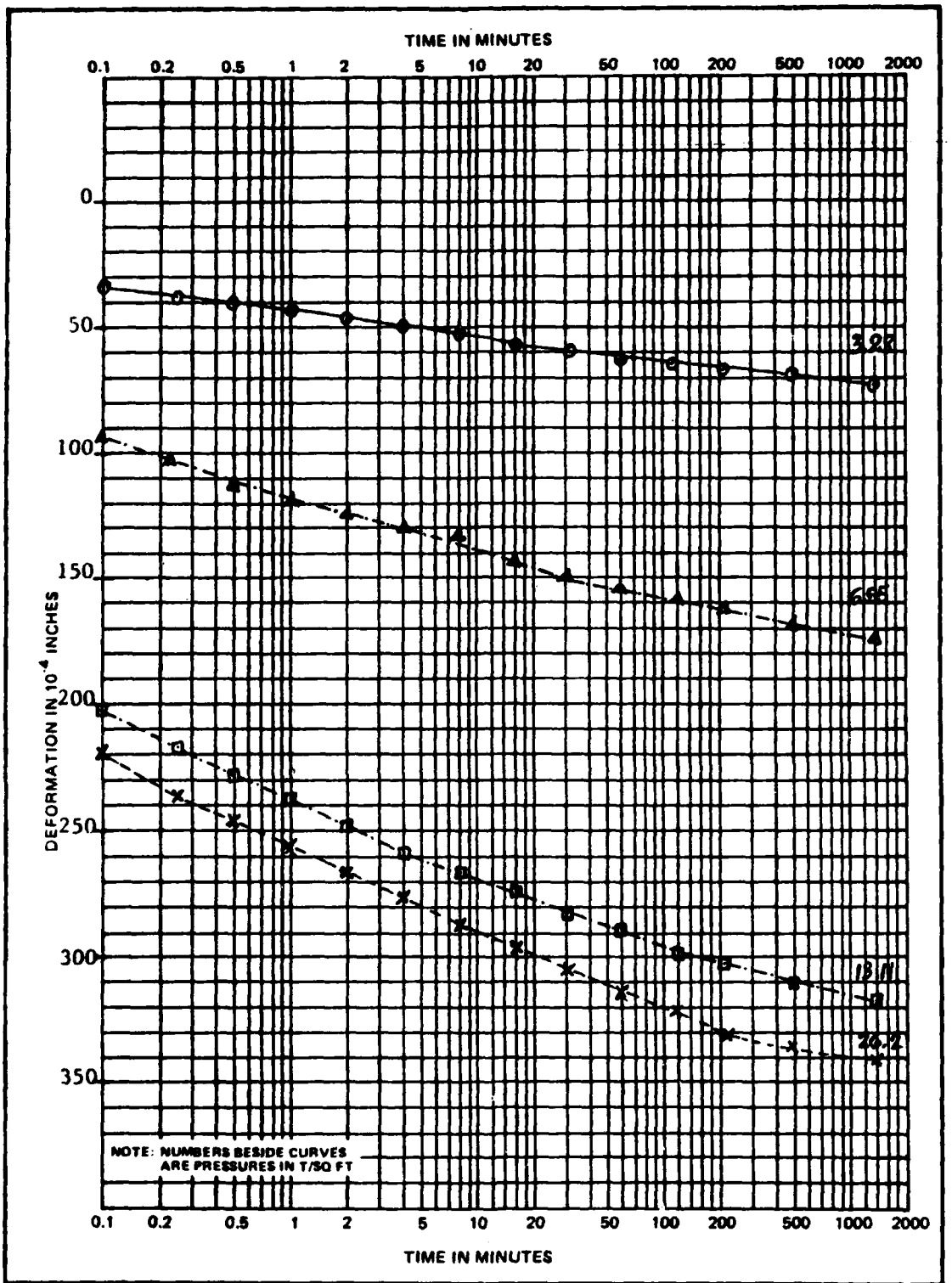
PLATE 38



PROJECT	Upper Santa Ana River			
AREA	Div. No. 97595 - 97596 Blend "C"			
BORING NO.	TH-21, 23	SAMPLE NO.	DEPTH EL. 13' & 15'	DATE October 1986

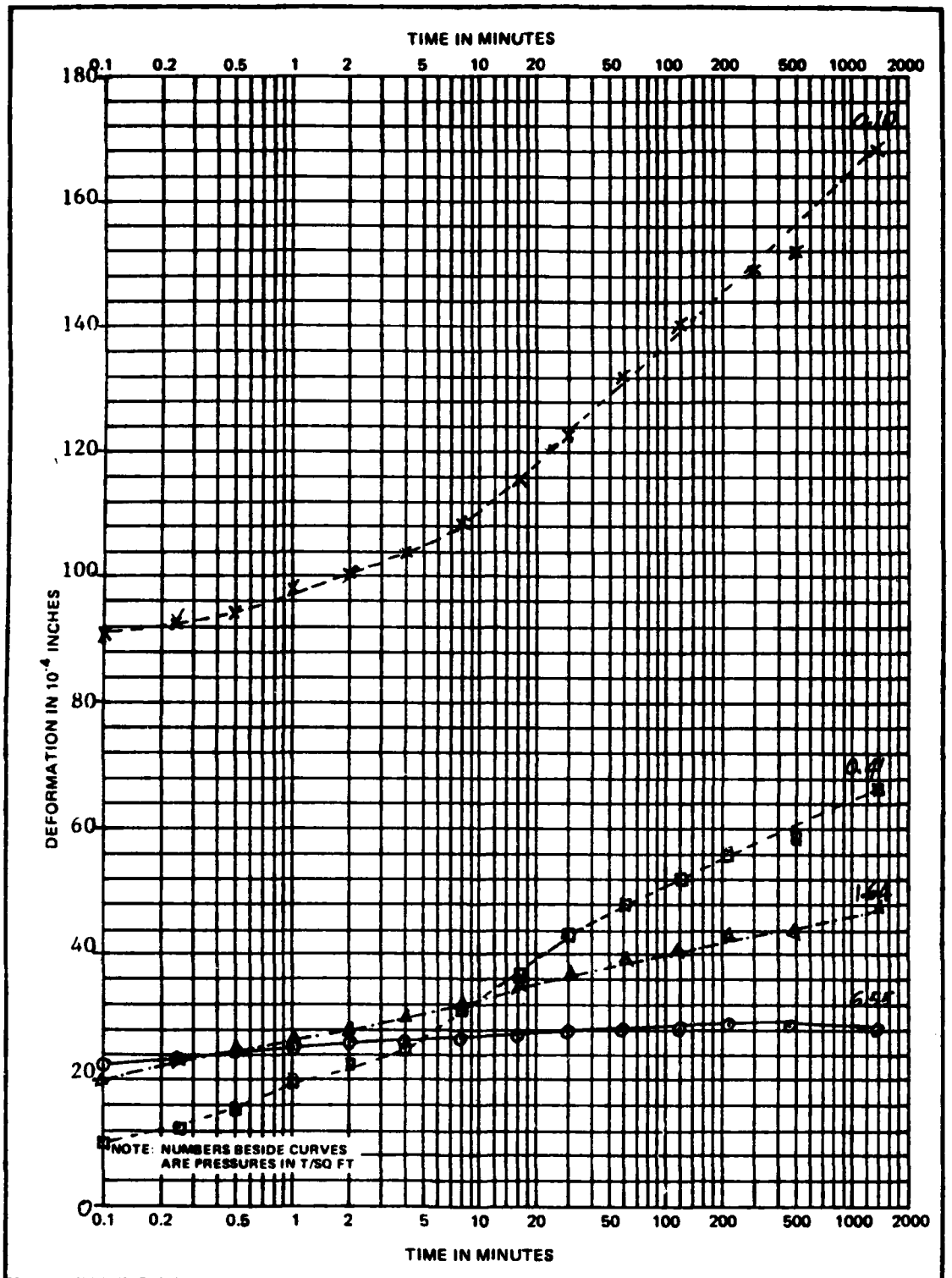
ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST--TIME CURVES** (TRANSLUCENT)

PLATE 39



PROJECT Upper Santa Ana River			
AREA Div. No. 97595 - 97596, Blend "C"			
BORING NO. TH-21, 23	SAMPLE NO.	DEPTH EL 13' & 15'	DATE October 1986
ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE	**CONSOLIDATION TEST–TIME CURVES** (TRANSLUCENT)		

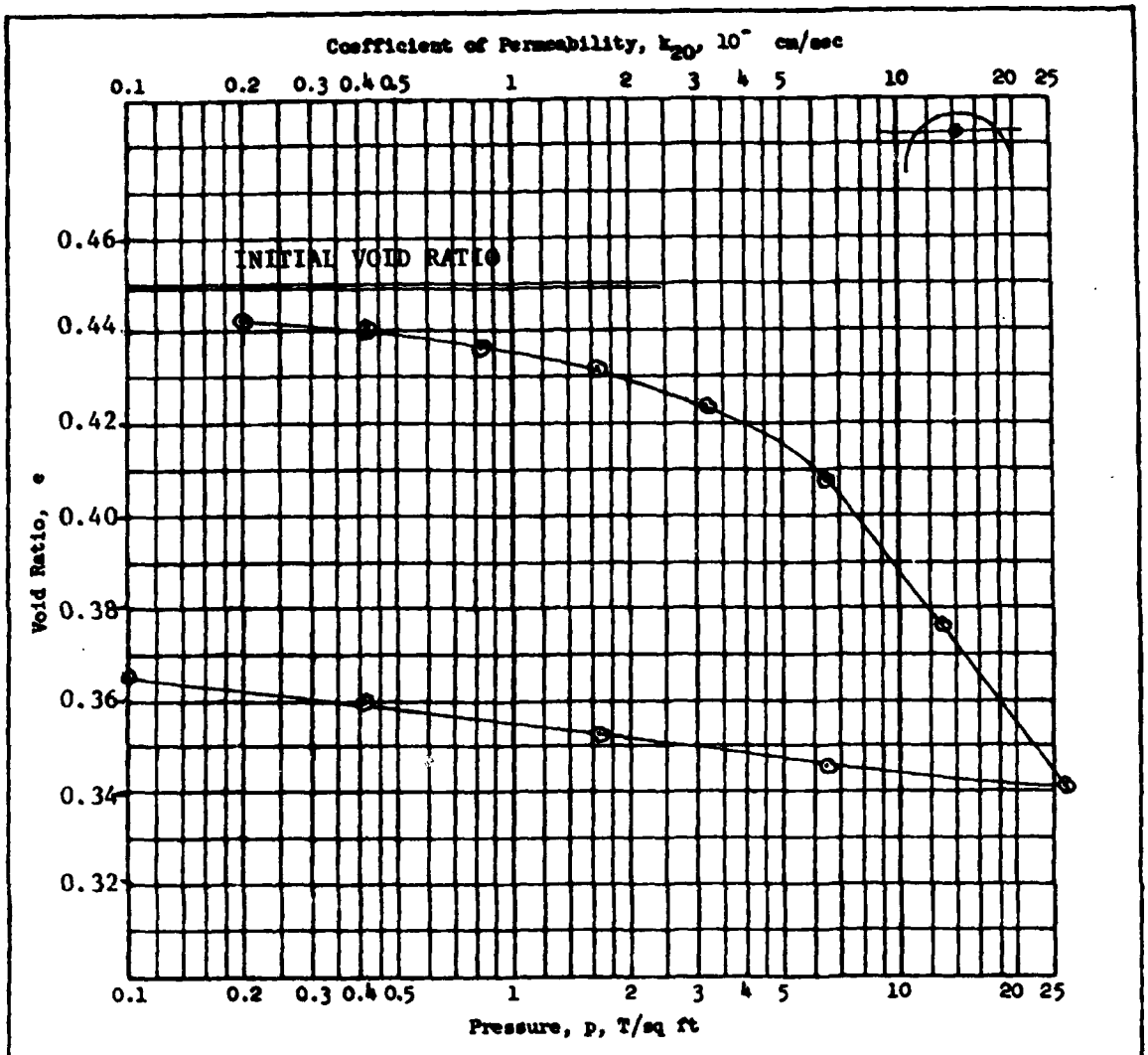
PLATE 40



PROJECT Upper Santa Ana River			
Div. No. 97595 - 97596 , Blend "C"			
BORING NO. TH-21, 23	SAMPLE NO.	DEPTH EL 13' & 15	DATE October 1986

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST–TIME CURVES** (TRANSLUCENT)

PLATE 41

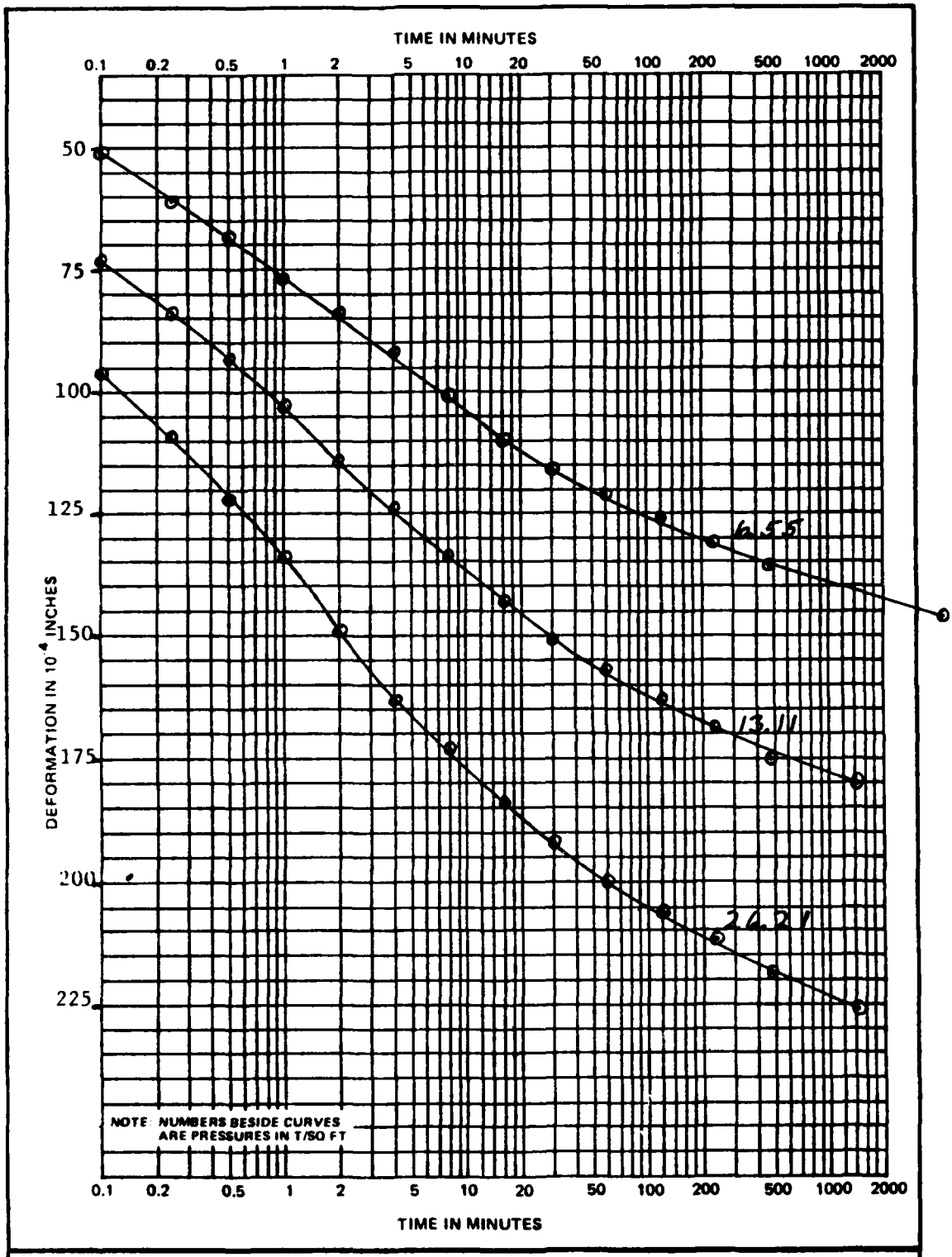


Type of Specimen Remolded			Before Test		After Test	
Diam 4.4 in.	Ht 1.00 in.		Water Content, w_o	12.2 %	w_f	12.6 %
Overburden Pressure, p_o T/sq ft			Void Ratio, e_o	0.449	e_f	0.376
Preconsol. Pressure, p_c T/sq ft			Saturation, S_o	74 %	S_f	91 %
Compression Index, C_c			Dry Density, γ_d	116.7 lb/ft^3		
Classification Clayey Sand(SC)			k_{20} at e_o =	x 10^{-} cm/sec		
LL 32	G_s 2.71		Project Upper Santa Ana River			
PL 18	D_{10}					
Remarks Remolded to 98% max. density of optimum water content			Area Impervious Core			
			Boring No. TH 21 & 23		Sample No. Comp. C	
			Depth El 0-15		Date July 1986	
Div. No. 96595 and 96596			CONSOLIDATION TEST REPORT			

ENG FORM 1 MAY 63 2090 PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT)

GPO 936-736

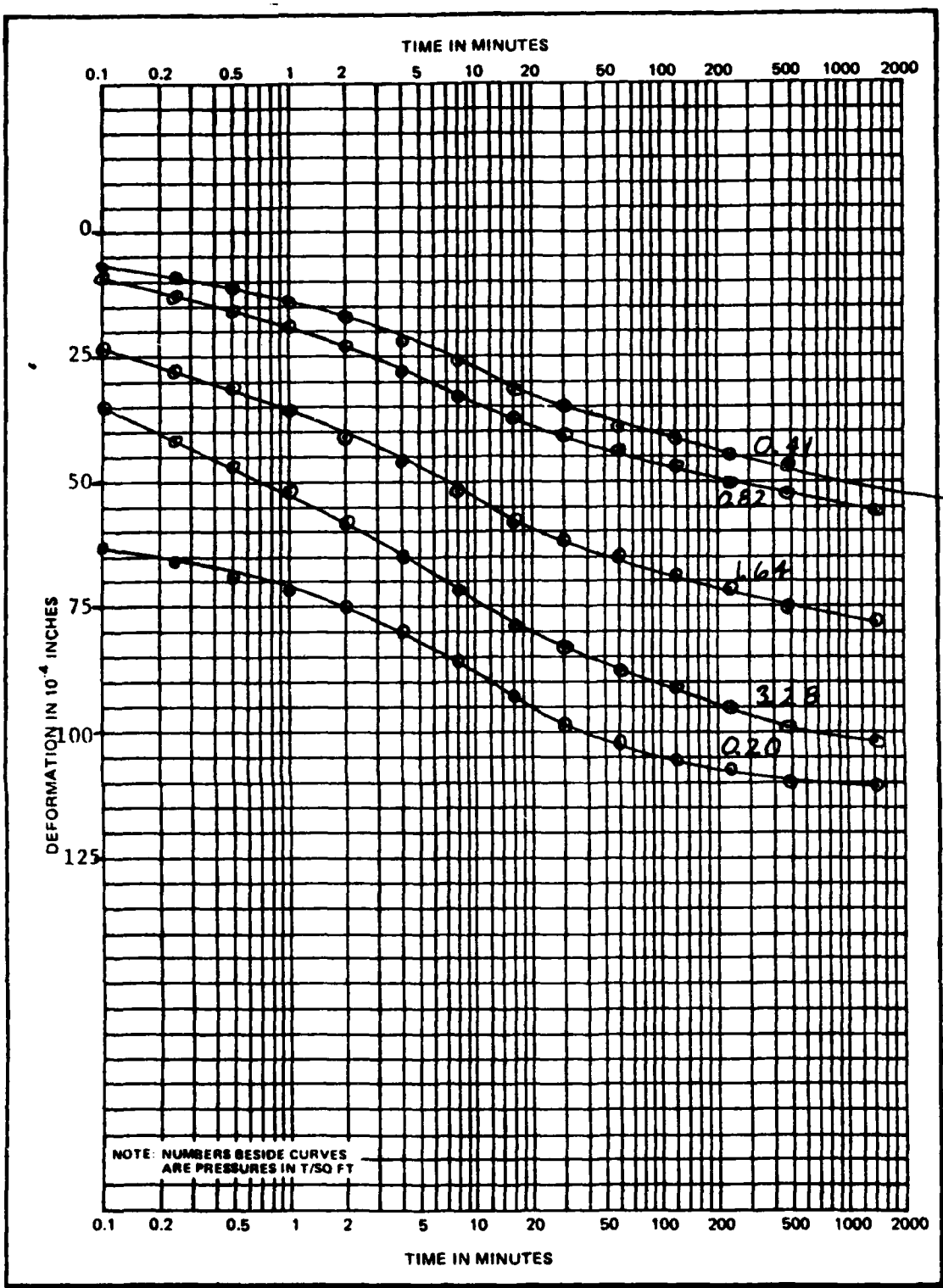
n 3424 PLATE 42



PROJECT Upper Santa Ana River			
AREA Div. No. 96595 & 96596			
BORING NO. TH-21&23	SAMPLE NO. Comp C	DEPTH EL 0-15	DATE July 1986

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST–TIME CURVES** (TRANSLUCENT)

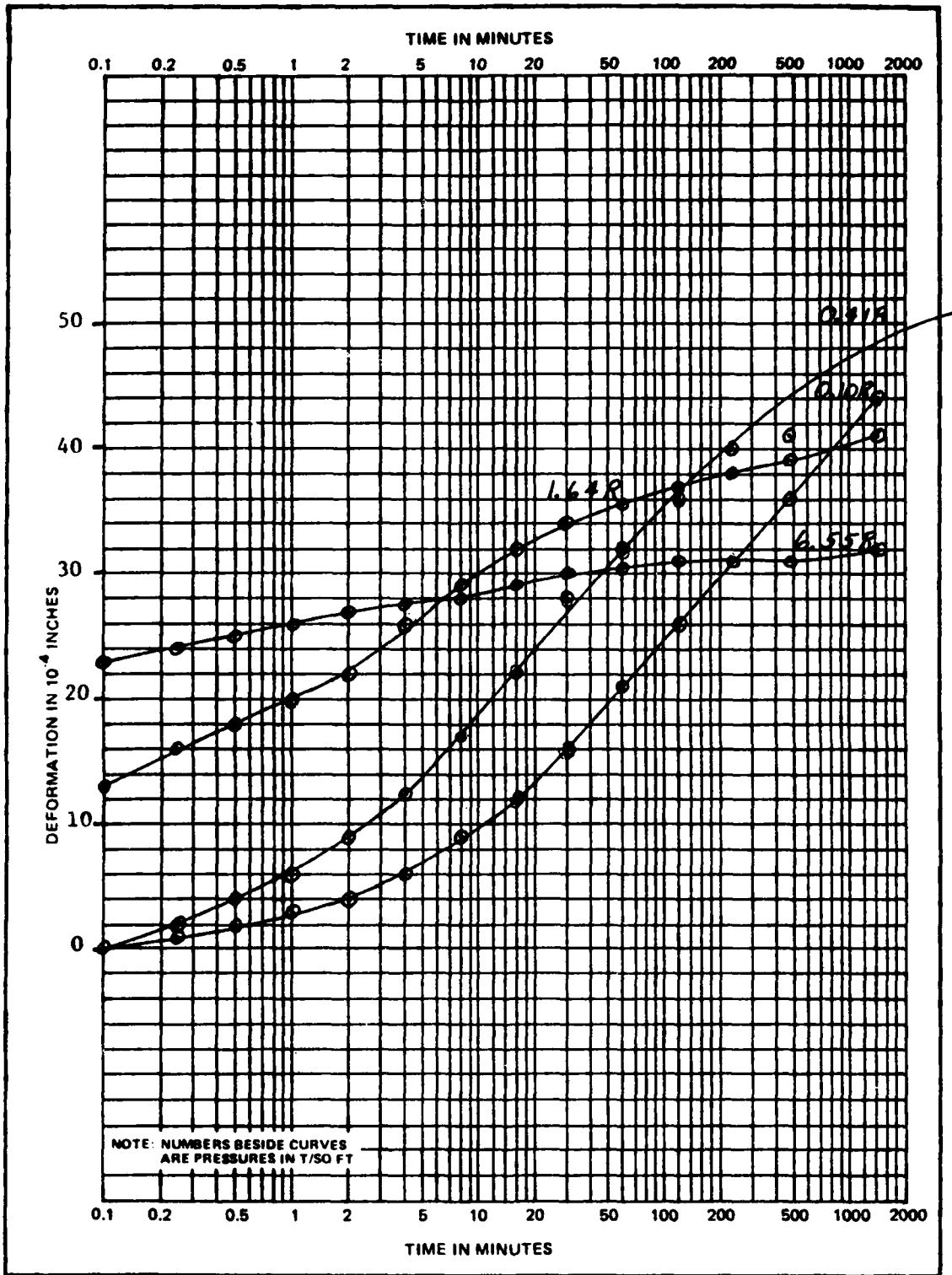
PLATE 43



PROJECT	Upper Santa Ana River			
AREA	Div. No. 96595 and 96596			
BORING NO.	TH-21&23	SAMPLE NO Comp C	DEPTH EL 0-15	DATE July 1986

ENG FORM 1 MAY 83 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST–TIME CURVES** (TRANSLUCENT)

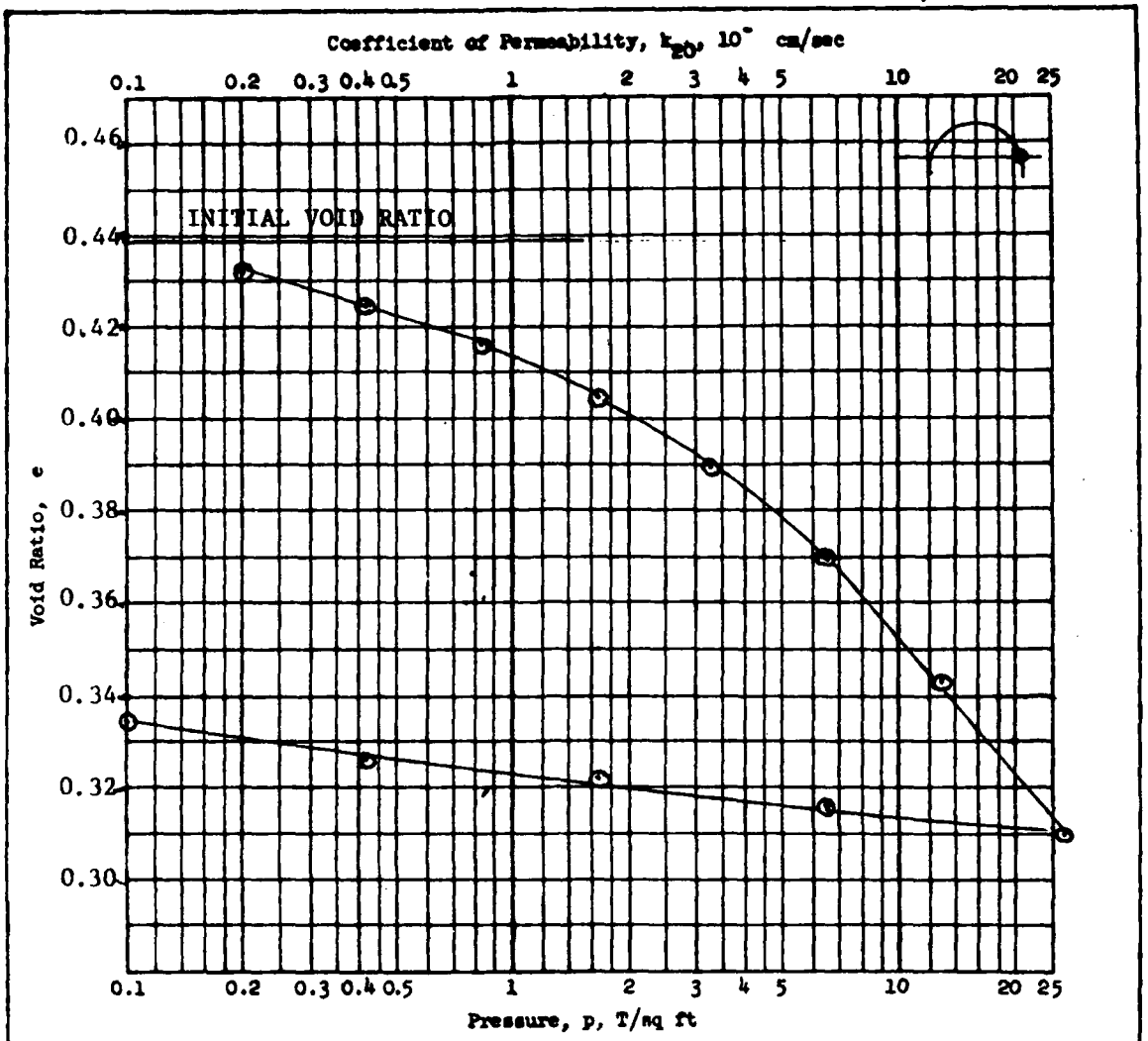
PLATE 44



PROJECT Upper Santa Ana River			
AREA Div. No. 96595 and 96596			
BORING NO. TH-21 & 23	SAMPLE NO. Comp C	DEPTH EL 0-15	DATE July 1986

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST—TIME CURVES** (TRANSLUCENT)

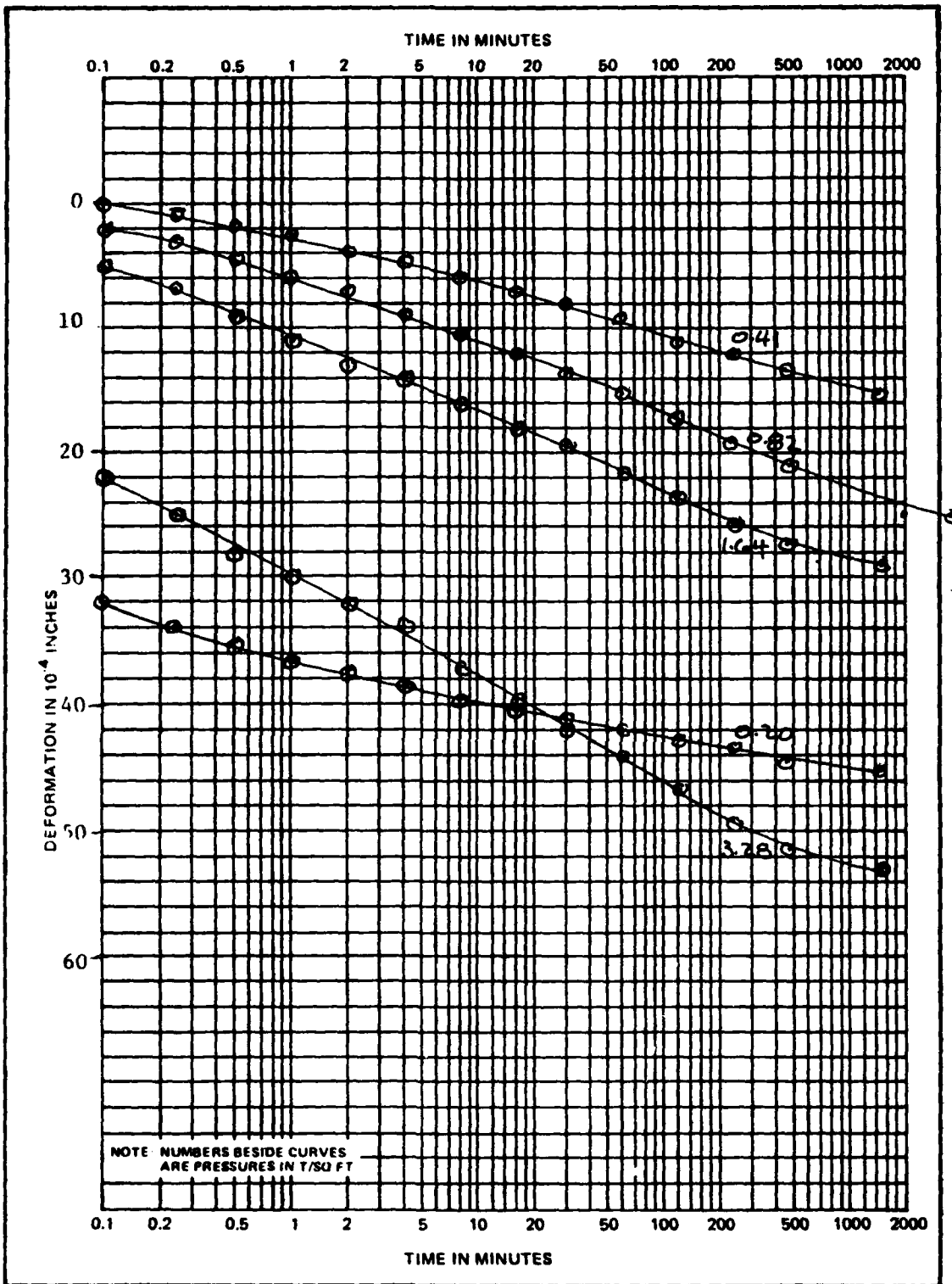
PLATE 45



Type of Specimen Remolded		Before Test		After Test	
Diam 4.4 in.	Ht 1.00 in.	Water Content, w_o	15.3 %	w_f	12.3 %
Overburden Pressure, Po T/sq ft		Void Ratio, e_o	0.449	e_f	0.343
Preconsol. Pressure, Pc T/sq ft		Saturation, S_o	92 %	S_f	97 %
Compression Index, C_c		Dry Density, γ_d	116.7 lb/ft^3		
Classification Clayey Sand(SC)		k_{20} at e_o =	× 10^{-} cm/sec		
LL 32	G_s 2.71	Project Upper Santa Ana River			
PL 18	D_{10}				
Remarks Remolded to 98% of max. density at 3% above optimum water content.		Area			
		Boring No. TH-21&23		Sample No. Comp. C	
		Depth El 0-15		Date	
Div. No. 96595 and 96596		CONSOLIDATION TEST REPORT			

ENG FORM 1 MAY 53 2090 PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT) 0 3424

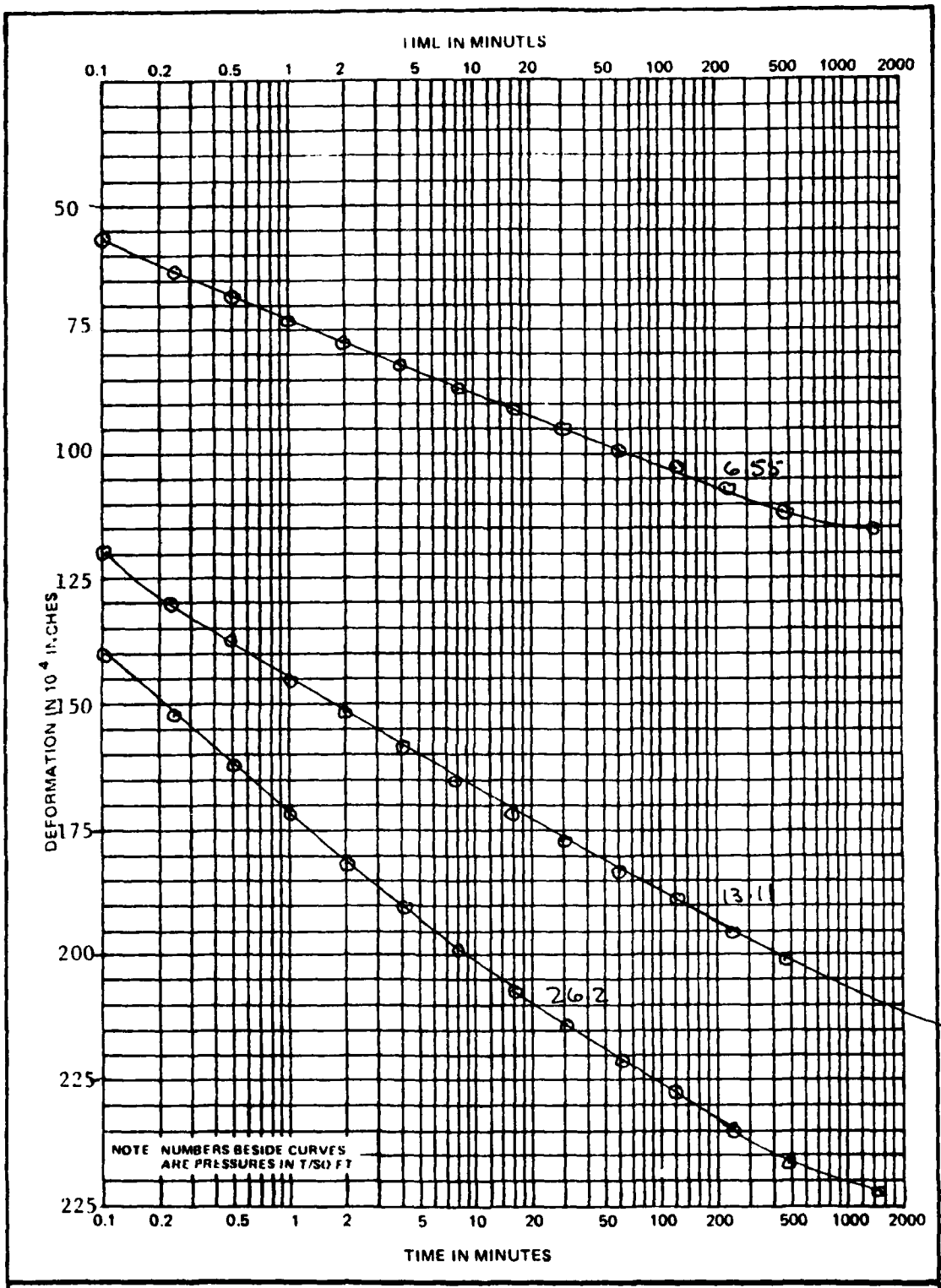
PLATE 46



PROJECT	Upper Santa Ana River			
AREA	Div. No. 96595 & 96596			
BORING NO. TH21 & 23	SAMPLE NO Comp. C	DEPTH EL 0-15	DATE July 1986	

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST–TIME CURVES** (TRANSLUCENT)

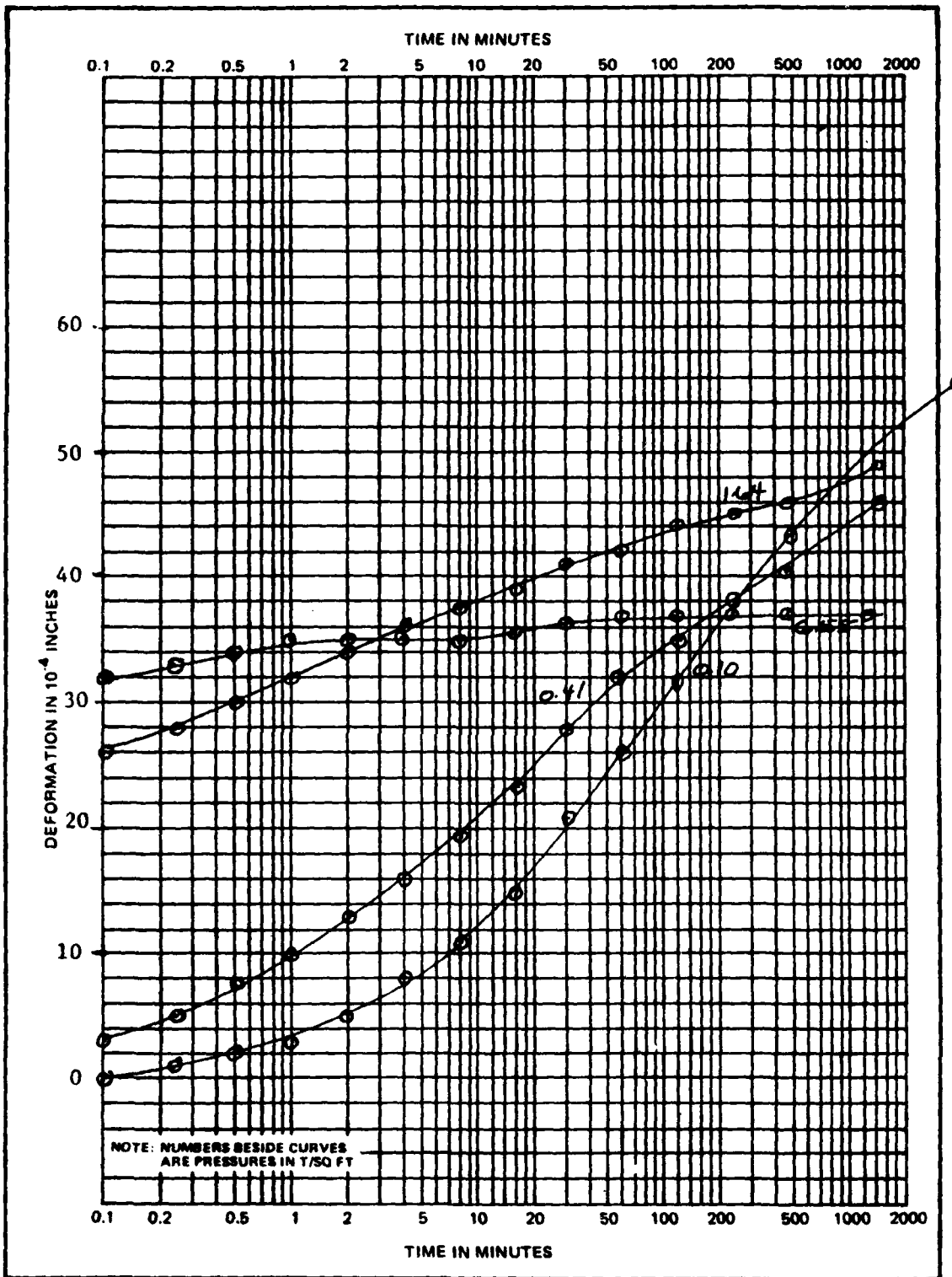
PLATE 47



PROJECT	Upper Santa Ana River		
AREA	Div. No. 96595 & 96596		
BORING NO. TH 21&23	SAMPLE NO Comp. C	DEPTH EL 0-15	DATE July 1986

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST–TIME CURVES** (TRANSLUCENT)

PLATE 48



PROJECT Upper Santa Ana River			
AREA Div. No. 96595 & 96596			
BORING NO. TH 21 & 23	SAMPLE NO Comp C	DEPTH EL 0-15	DATE July 1986

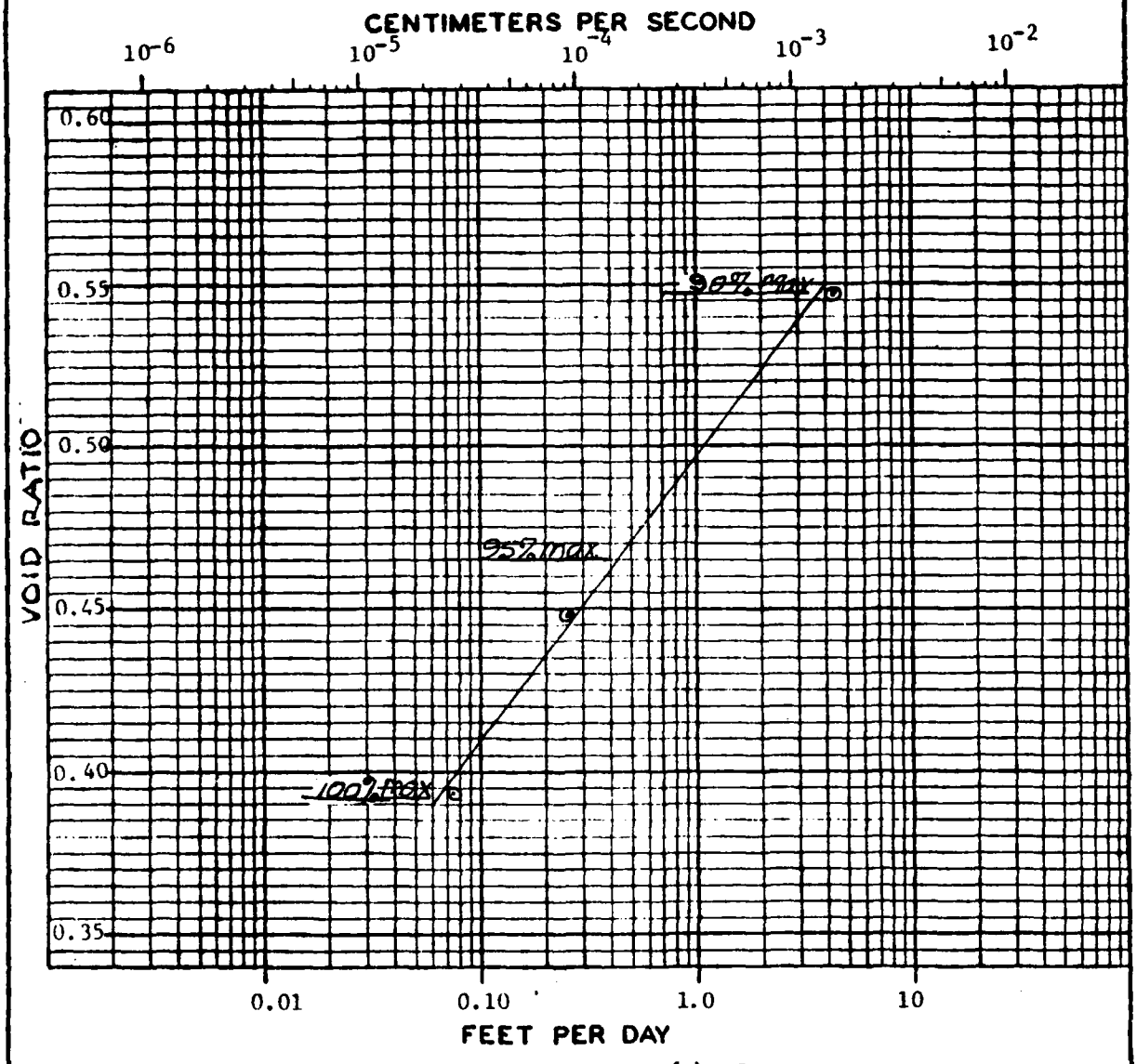
ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST–TIME CURVES** (TRANSLUCENT)

PLATE 49

CORPS OF ENGINEERS — U. S. ARMY

SOUTH PACIFIC DIVISION LABORATORY

PERMEABILITY



COEFFICIENT OF PERMEABILITY (K) AT 20°C

REMARKS:

CURVE	SPECIMEN DIAM. IN.	HT. IN.	MAX. PARTICLE	CONDITION
	6.0	12.35	1-in.	Remolded to optimum

DISTRICT: Los Angeles

PROJECT: Upper Santa Ana River

CURVE	DIV. NO.	HOLE NO.	F.S. NO.	DEPTH FROM	DEPTH TO
Comp	96588 96589	TH-85-9		Blend	"B"

TESTED	COMPUTED	DRAWN	CHECKED
FN			

SPD FORM 4 OCT 55 67 (CIVIL) REPL ...n 67 (CIVIL), 20 JULY 49, WHICH MAY BE USED.

PLATE 50

U.S. ARMY ENGINEER DIVISION LABORATORY -- SOUTH PACIFIC DIVISION

SOIL TEST RESULT SUMMARY

PROJECT Upper Santa Ana River **DATE** May 1986

Division Serial No.	Hole No.	Field Sample No.	Depth Or Elevation		Laboratory Descriptive Classification	Mechanical Analysis-% Finer											Liquid Limit	Plasticity Index	Field Moist. %
						Gravel					Sand					Fines			
			From	To		3/2	1 1/2	3/4	1/2	3/8	#4	#10	#40	#60	#100	#200			
96586	TH-85-4		4		Sandy Clay (CL-ML)						100	96	78	70	61	50	27	7	
96587	TH-85-9		6		Clayey Sand (SC-SM)		100 99	99	99	99	97	93	69	58	48	35	26	7	
96588	TH-85-9		15		Silty Sand(SM)		100	99	99	99	97	93	70	59	49	36	23	3	
96589	"		26		Silty Sand(SM)	100	99 99	98	96	94	89	82	49	37	27	17		NP	
96590	TH-85-10		18	21	Silty Sand(SM)	100	99 98	97	96	96	94	85	55	45	37	27	22	3	
96591	TH-85-11		20		Sandy Silt(ML)		100	99	99	98	96	94	79	72	64	53	22	2	
96592	TH-85-14		8		Silty Sand (SP-SM)				100	99	98	89	49	36	23	9	24	3	
96593	"		19		Silty Sand(SM)		100 99	99	98	97	94	87	62	52	41	28		NP	
96594	"		44		Clayey Sand (SC-SM)				100	99	98	95	73	63	53	40	25	6	
96595	TH-85-21		0	13	Clayey Sand(SC)	100	99 99	99	98	98	97	92	73	65	57	47	31	13	
96596	TH-85-23		0	15	Silty Clayey Sand(SC)	100 98	98 96	95	94	93	88	78	47	38	31	23	29	8	

PLATE 1

SPD Form 66A
1 May 83

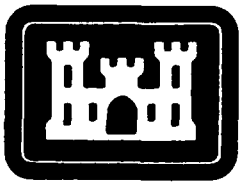
Division Laboratory

Sausalito, California

REPORT
OF
SOIL TESTS

UPPER SANTA ANA-SEVEN OAKS DAM
PERVIOUS BORROW MATERIAL

MAY 1987



US Army Corps
of Engineers
South Pacific Division

REPORT
OF
SOIL TESTS

UPPER SANTA ANA-SEVEN OAKS DAM
PERVIOUS BORROW MATERIAL

MAY 1987

AUTHORIZATION

1. Testing services were authorized by the Los Angeles District per DA Form 2544 No. E86 87 0081 dated 13 April 1987.

SAMPLES

2. On 25 November 1986, fifty-five samples in barrels were received at the Laboratory. Identification of the samples is shown on the Soil Test Result Summary, plate 1.

TESTING PROGRAM

3. The program was in accordance with the test request.

TESTS AND TEST METHODS

4. Grain-size Analysis, Specific Gravity, Triaxial Compression, Compaction and Permeability tests conformed to the procedure described in Engineer Manual EM 1110-2-1906, "Laboratory Soil Testing," 30 November 1970.

CLASSIFICATION

5. The soil was classified in accordance with "The Unified Soil Classification System," TM No. 3-357, Appendix A, April 1980.

RESULTS

6. Results of tests are shown on the following plates:

Subject	Plate No.
Soil Test Result Summary	1
Gradation Curves	2-3
Permeability	4
Triaxial Compression Test Report	
"$\bar{R}$" Tests	5-5A
"S" Tests	5B
Compaction Test Report	6-7

COMMENTS

7. The batch moisture contents for the two uppermost points on the compaction curve were higher than the resultant moisture contents due to bleeding during vibration. Even though testing started with batch moisture contents of 7½% and 8½%, both resulted in final moistures of 6.8%.

U.S. ARMY ENGINEER DIVISION LABORATORY -- SOUTH PACIFIC DIVISION

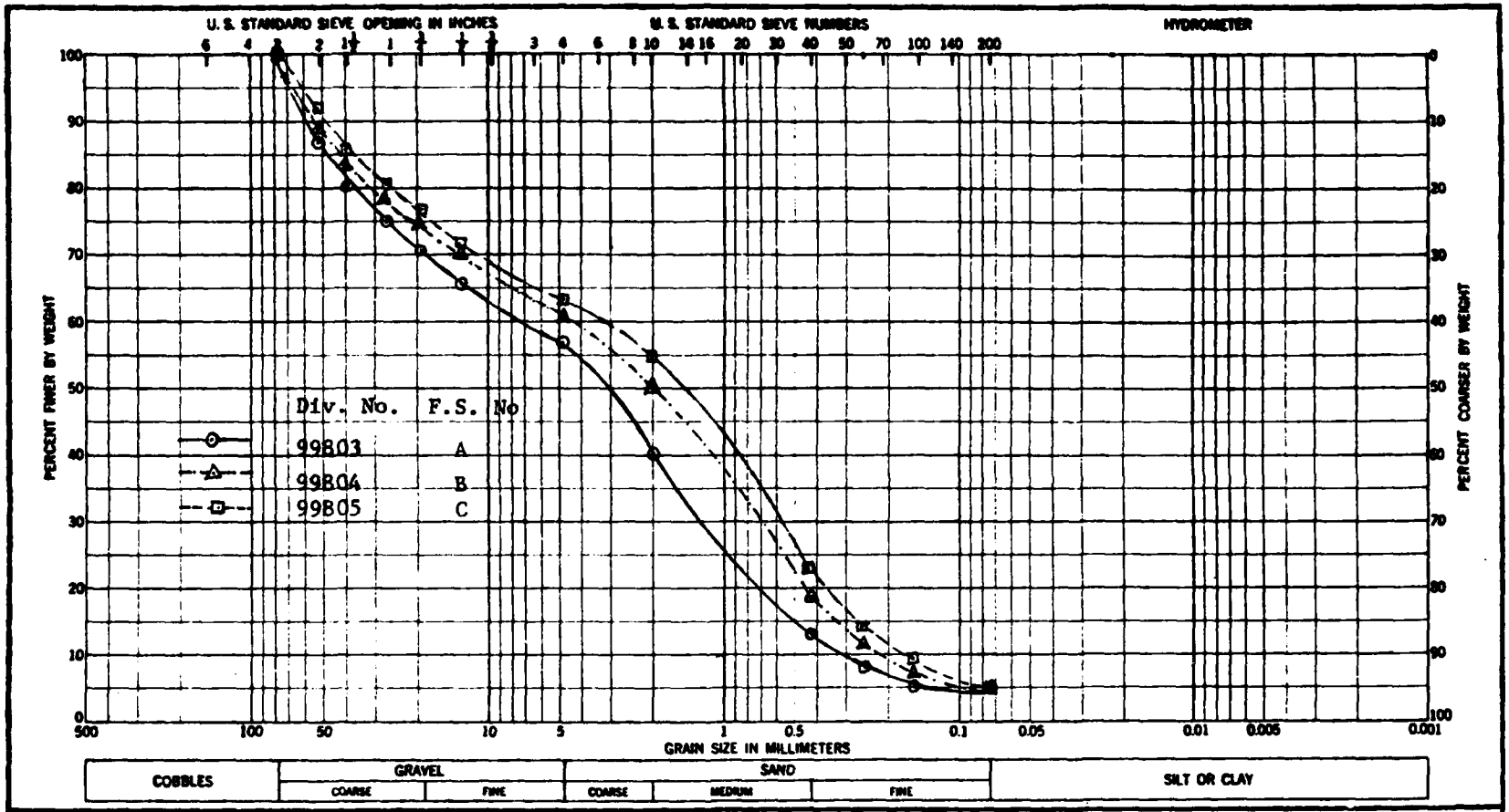
SOIL TEST RESULT SUMMARY

PROJECT Upper Santa Ana River - Seven Oaks Dam (Pervious Borrow Materials) **DATE** January 1987

Division Serial No.	Hole No.	Field Sample No.	Depth Or Elevation From	Depth Or Elevation To	Laboratory Descriptive Classification	Mechanical Analysis-% Finer: Gravel 3"	1 1/2"	3/4"	1/2"	3/8"	Sand #4	#10	#40	#60	#100	Fines #200	Liquid Limit	Plasticity Index	Field Moist. %
99803	TP-3	A			Gravelly Sand (SP)	100 87	81 75	71	66	66	57	40	13	.8	5	4			0.97
99804	"	B			Gravelly Sand (SP)	100 88	83 78	74	70	70	61	50	18	12	7	5			0.92
99805	"	C			Gravelly Sand (SP)	100 92	86 81	77	72	72	63	55	23	14	9	5			1.89
99806	"	D			Gravelly Sand (SP)	100 88	83 78	74	70	70	60	50	17	11	7	5			1.92
99807	"	E			Gravelly Sand (SP)	100 88	82 77	73	68	68	58	52	20	13	8	5			1.68
					Specific Gravity														
					-4 +4														
99803		A			2.59 2.66														
99804		B			2.60														
99805		C			2.64														
99806		D			2.61														

PLATE 1

SPD Form 66A
1 May 83



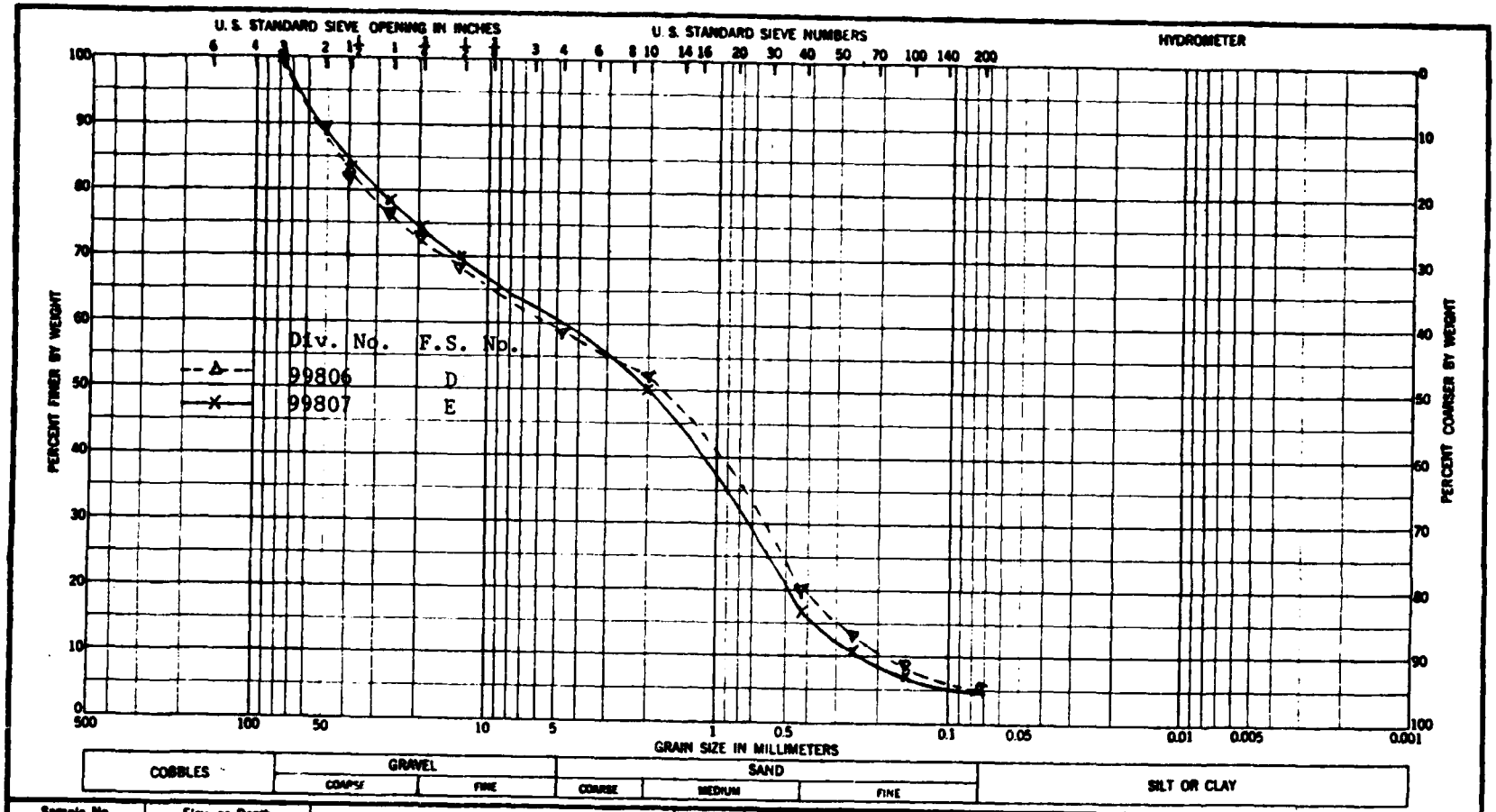
Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
A		Gravelly Sand(SP)	0.97			
B		Gravelly Sand(SP)	0.92			
C		Gravelly Sand(SP)	1.89			

GRADATION CURVES

Project Upper Santa Ana River
Seven Oaks Dam
Area Pervious Borrow Materials
Boring No. TP-3
Date January 1987

PLATE 2

ENG FORM 1 MAY 63 2087



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
D		Gravelly Sand(SP)	1.92			
E		Gravelly Sand(SP)	1.68			

GRADATION CURVES

Project Upper Santa Ana River
Seven Oaks Dam
Area Pervious Borrow Materials
Boring No. TP-3
Date January 1987

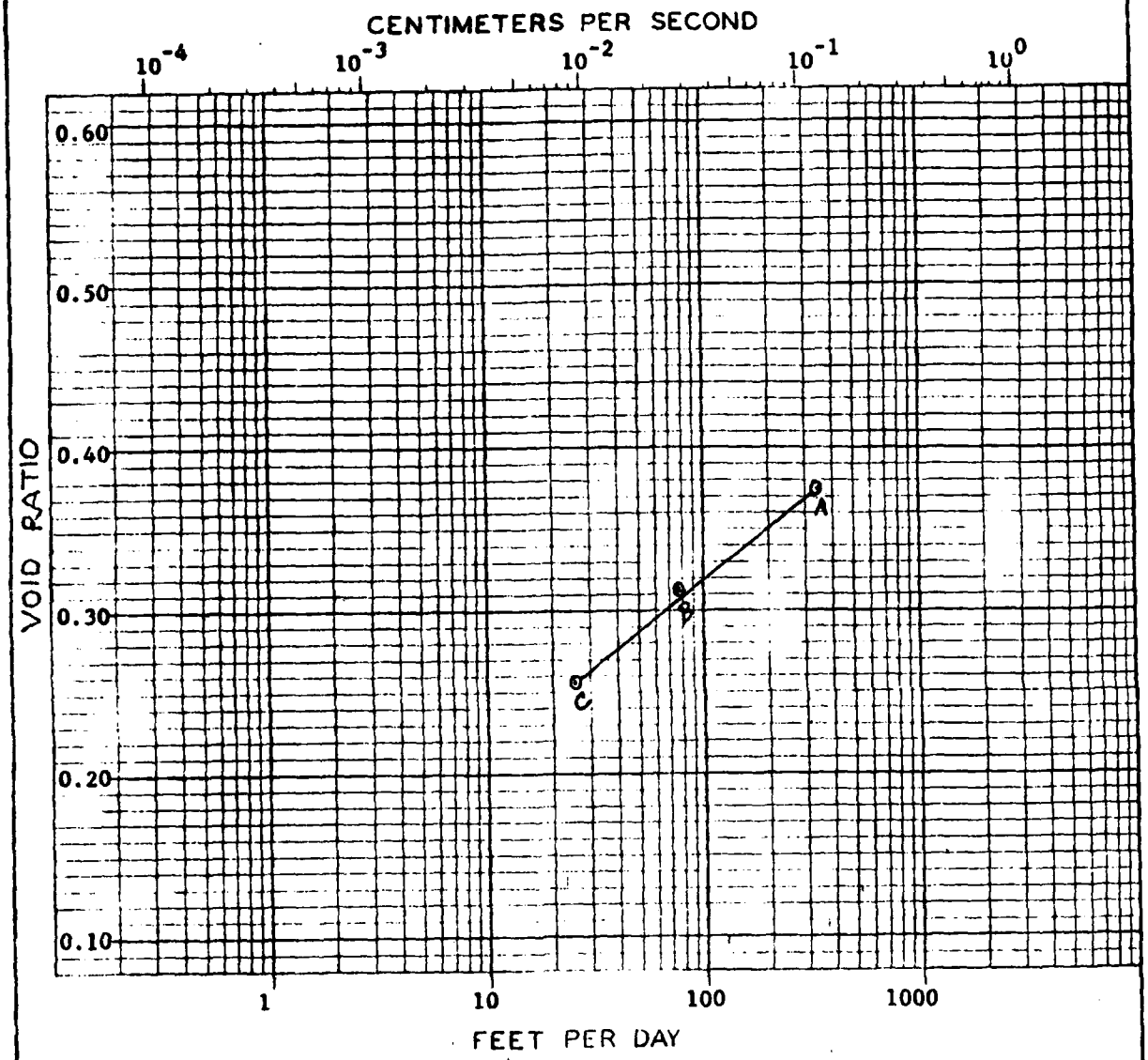
PLATE 3

ENG FORM 1 MAY 62 2087

CORPS OF ENGINEERS — U. S. ARMY

SOUTH PACIFIC DIVISION LABORATORY

PERMEABILITY



COEFFICIENT OF PERMEABILITY (K)

REMARKS: Classification: Gravelley Sand (SP) γmax=132 pcf Opt Moist=6.8%

CURVE Point	SPECIMEN DIAM. IN.	HT. IN.	MAX. PARTICLE	CONDITION Remolded
A	12"	27.63	3"	90.0% γmax.
B	"	27.63	"	94.1% γmax.
C	"	27.59	"	98.7% γmax.

DISTRICT: Los Angeles

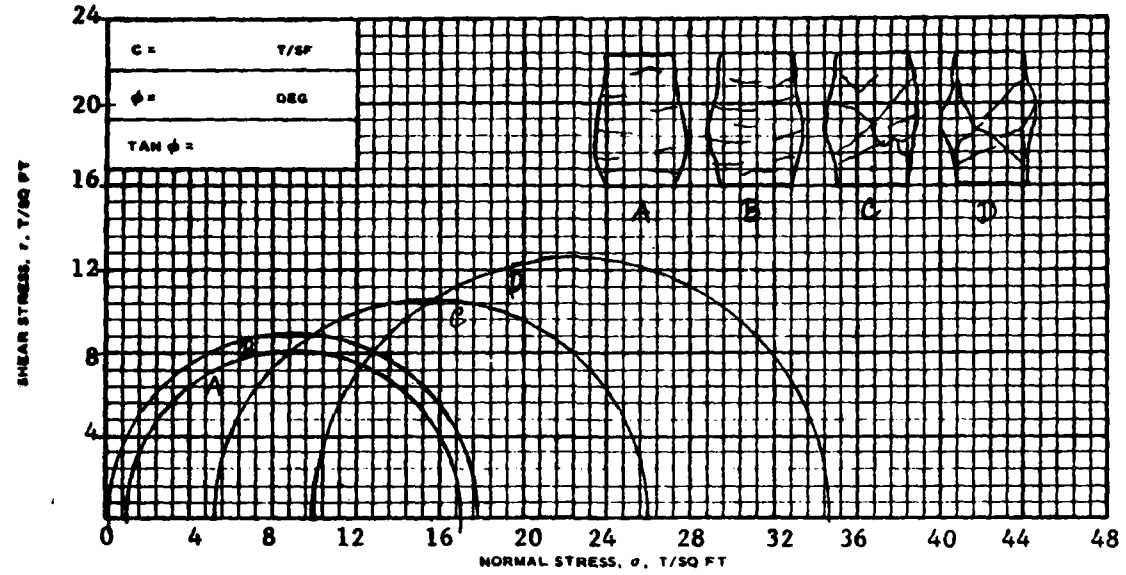
PROJECT: SEVEN OAKS DAM Pervious

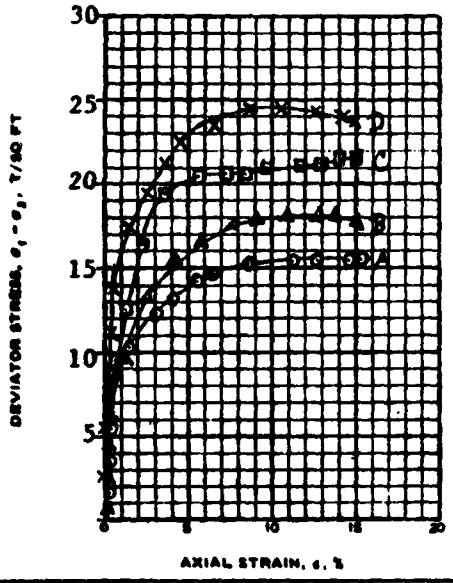
CURVE	DIV. NO.	HOLE NO.	F.S. NO.	DEPTH FROM	DEPTH TO
	99803, 4	TP-3	A,B		

TESTED	COMPUTED	DRAWN	CHECKED
FN	FN	CRK	

SPD FORM 6 OCT 55 67 (CIVIL) REPLACES PSD 67 (CIVIL), 20 JULY 49, WHICH MAY BE USED.

PLATE 4





SPECIMEN NO.			A	B	C	D
INITIAL	WATER CONTENT, %	w_o	7.9	8.0	8.3	8.5
	DRY DENSITY LB/ CU FT	γd_o	126.0	126.0	125.6	125.4
	SATURATION, %	S_o	70	70	72	73
	VOID RATIO	e_o	0.297	0.298	0.302	0.304
BEFORE SHEAR	WATER CONTENT, %	w_c	7.4	8.2	7.5	8.2
	DRY DENSITY LB/CU FT	γd_c	137	127	126	126
	SATURATION, %	S_c	100	74	66	72
	VOID RATIO	e_c	0.194	0.290	0.295	0.297
	FINAL BACK PRESSURE, T/SQ FT	u_b	6.12	6.12	6.12	6.12
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	1.0	2.5	5.0	10.0
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	15.56	18.03	21.83	24.26
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	414	340	407	340
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$	-	-	-	-
INITIAL DIAMETER, IN.		D_o	12.07	12.07	12.07	12.07
INITIAL HEIGHT, IN.		H_o	27.65	27.65	27.65	27.65

CONTROLLED- Strain TEST

DESCRIPTION OF SPECIMENS Gravelly Sand (SP)

LL	PL	PI NP	G_s 2.62	TYPE OF SPECIMEN Remolded	TYPE OF TEST "R"

REMARKS:

Div. No. 99803 & 804

PROJECT UPPER SANTA ANA RIVER

Seven Oaks Dam

BORING NO. TP-3 SAMPLE NO. A & B

DEPTH/ELEV Pervious Borrow Materials

LABORATORY SPDL DATE May 1987

TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE

PLATE 5

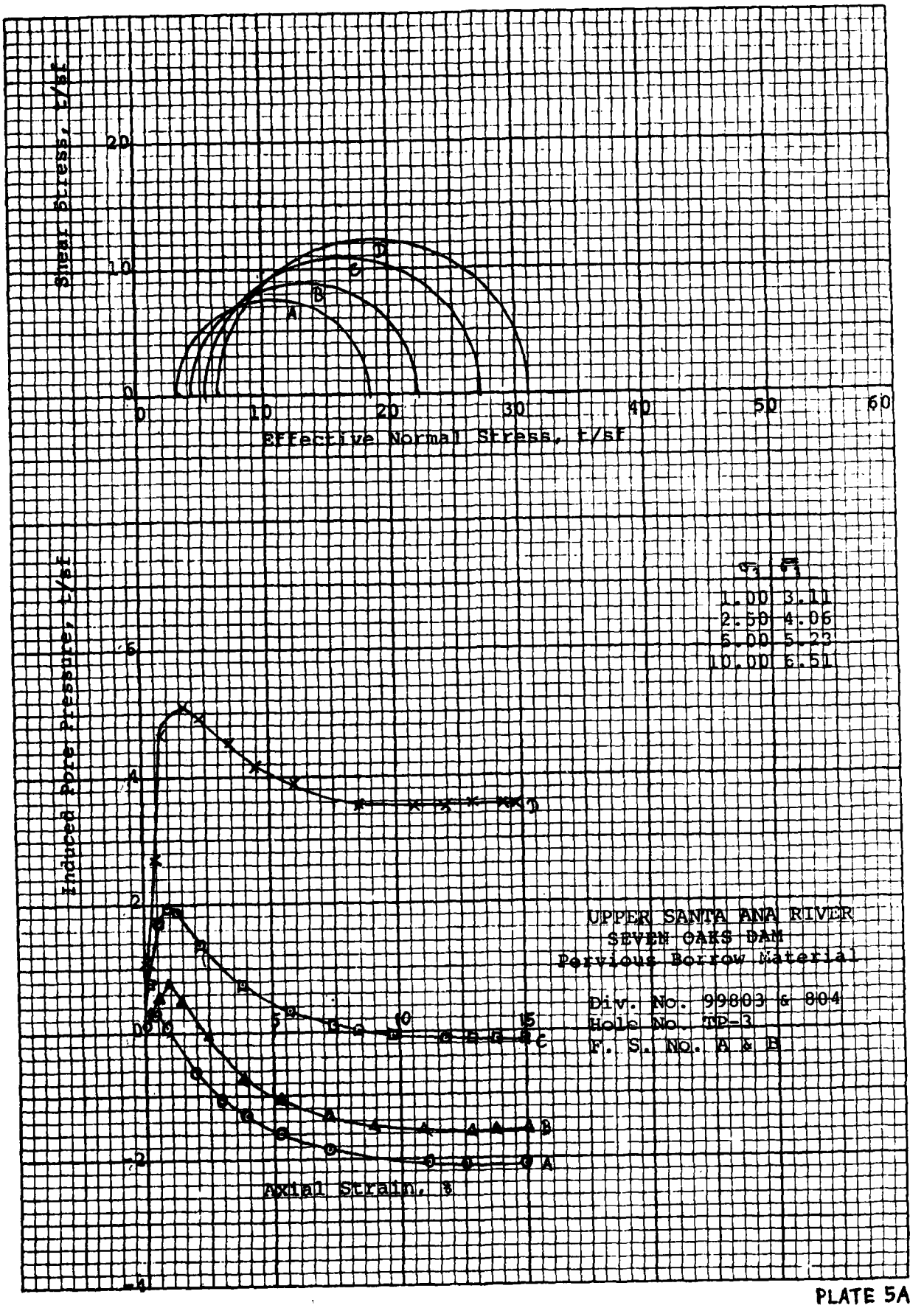
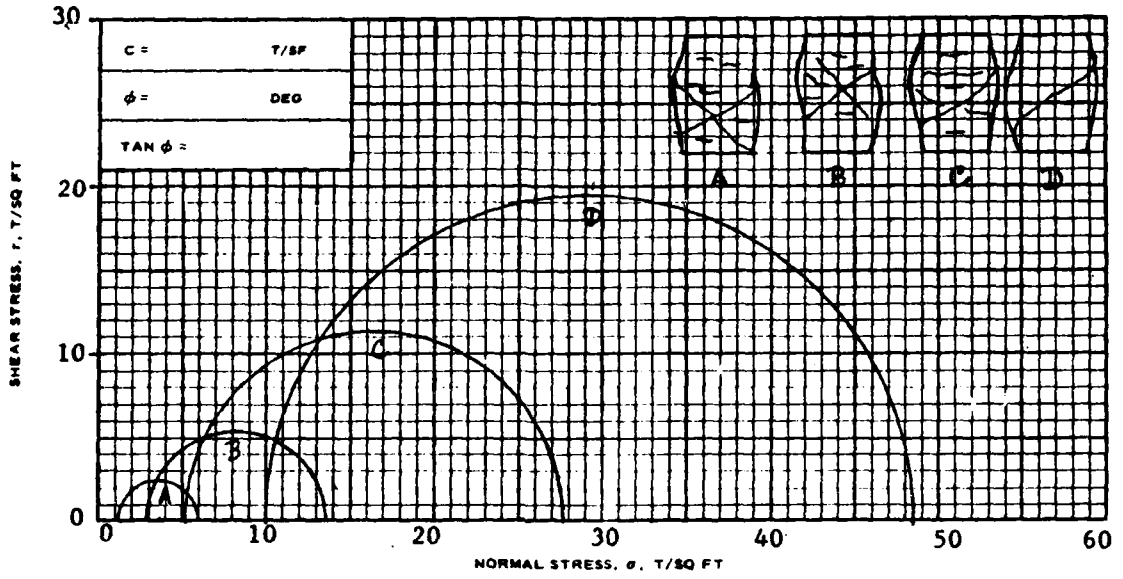
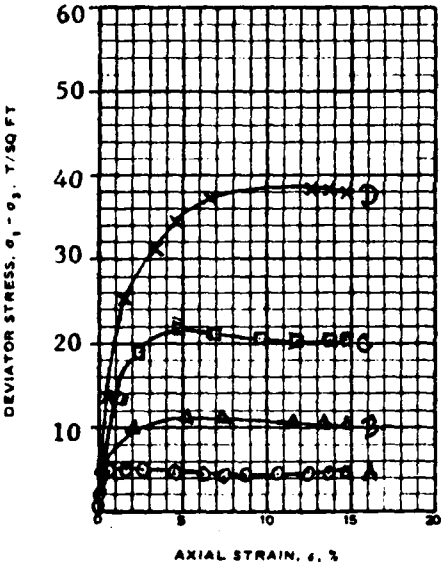

Shear Stress, t/sf
Effective Normal Stress, t/sf
Excess Pore Pressure, t/sf
Axial Strain, %
σ_3 $\bar{\sigma}_3$
1.00 3.11
2.50 4.06
5.00 5.23
10.00 6.51
UPPER SANTA ANA RIVER
SEVEN OAKS DAM
Pervious Borrow Material
Div. No. 99803 & 804
Hole No. TP-3
F. S. No. A & B

PLATE 5A





SPECIMEN NO.			A	B	C	D
INITIAL	WATER CONTENT, %	w_o	8.2	8.2	8.2	8.2
	DRY DENSITY LB/CU FT	γd_o	124.8	124.8	124.7	124.7
	SATURATION, %	S_o	67	67	67	67
	VOID RATIO	e_o	0.325	0.325	0.326	0.326
BEFORE SHEAR	WATER CONTENT, %	w_c	8.0	8.1	9.2	9.2
	DRY DENSITY LB/CU FT	γd_c	125.0	125.3	125.0	125.2
	SATURATION, %	S_c	65	67	75	76
	VOID RATIO	e_c	0.323	0.320	0.323	0.320
	FINAL BACK PRESSURE, T/SQ FT	u_b	6.12	6.12	6.12	6.12
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	1.0	2.5	5.0	10.0
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	5.38	11.28	22.35	38.38
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	140	160	220	320
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$	-	-	-	-
INITIAL DIAMETER, IN.		D_o	12	12	12	12
INITIAL HEIGHT, IN.		H_o	27.63	27.64	27.61	26.61

CONTROLLED- Strain TEST

DESCRIPTION OF SPECIMENS Gravelly Sand (SP)

LL	PL	PI	G_s
		NP	2.65

TYPE OF SPECIMEN Remolded TYPE OF TEST "S"

REMARKS: Div. No. 99805 & 806

PROJECT UPPER SANTA ANA RIVER
Seven Oaks Dam-Pervious Borrow Material

BORING NO. TP-3 SAMPLE NO. C & D

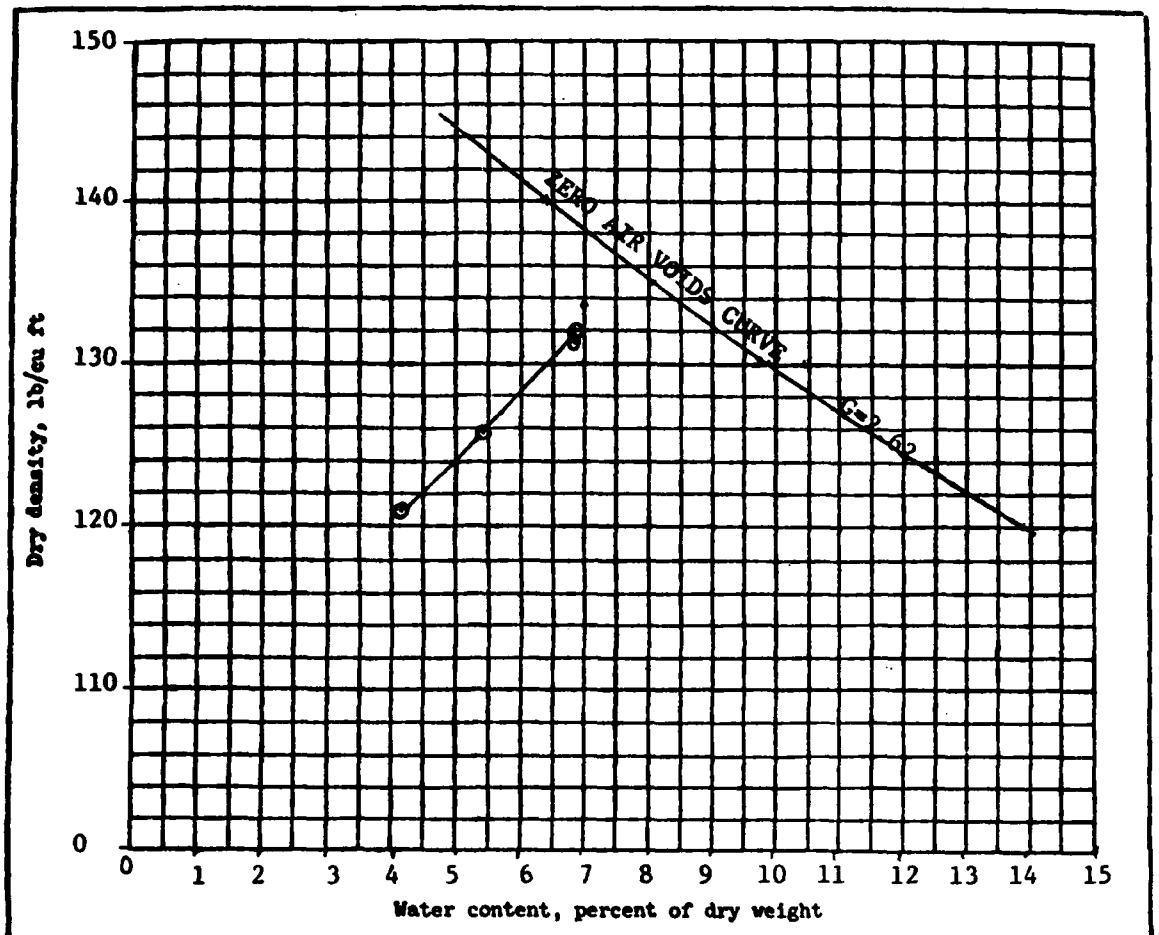
DEPTH/ELEV

LABORATORY SPDL DATE May 1987

TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089 REV JUNE 1970 PREVIOUS EDITION IS OBSOLETE

PLATE 5B



Max-Min compaction test

0 blows per each of 2 layers, with 2 psi surcharge rammer and

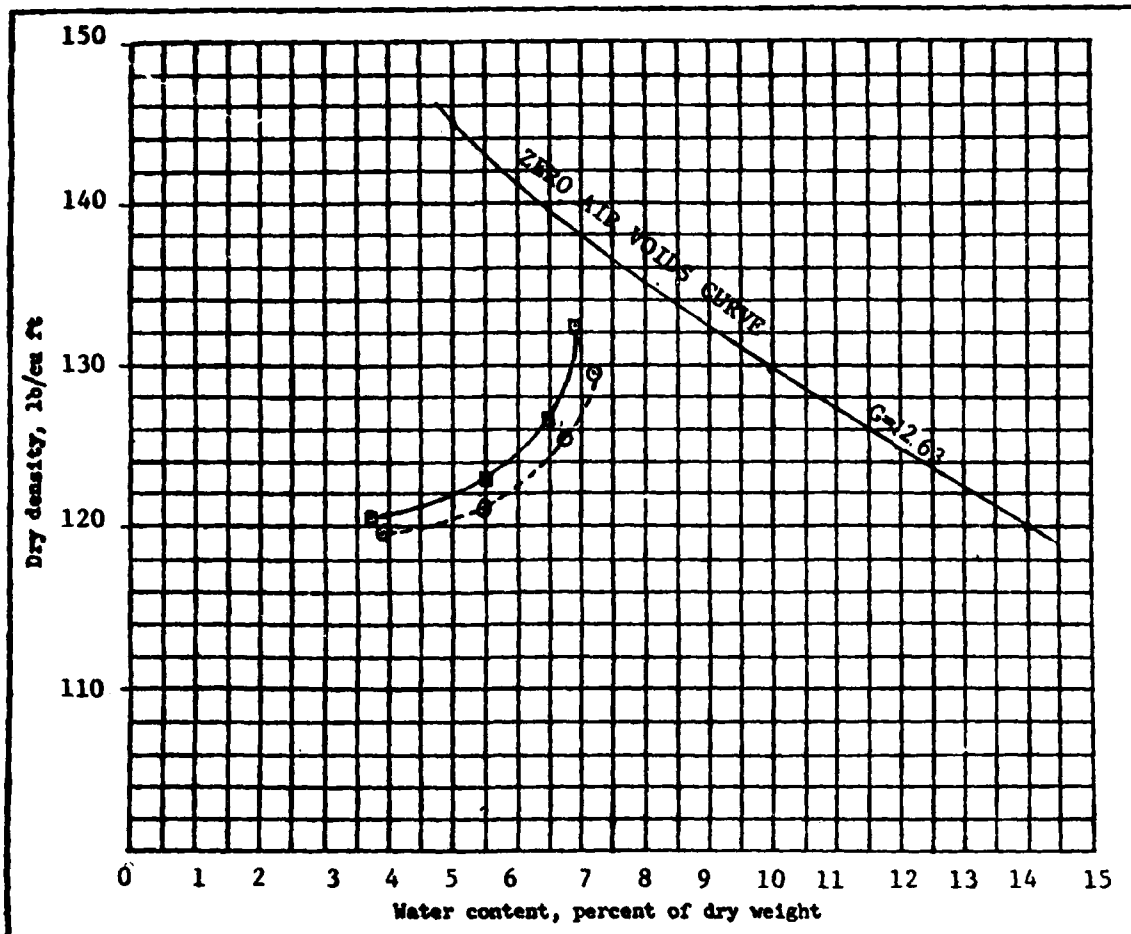
0 inch drop. 11 inch diameter mold

Sample No.	Elev or Depth	Classification	G	LL	PL	% > No. 4	% > 3/4 in.
Composite							
99803 & 99804		Gravelly Sand(SP)	2.69	NP	NP	41	27

Sample No.	99803 & 99804			
Natural water content, percent	Unknown			
Optimum water content, percent	6.8			
Max dry density, lb/cu ft	132			

Remarks	Project UPPER SANTA ANA RIVER- Seven Oaks Dam
Div. No. 99803 & 99804	Pervious Borrow Materials
	Area
*See remark on summary	Boring No. TP-3 / Date May 1987
sheet.	**COMPACTION TEST REPORT**

ENG FORM 1 JUN 65 2091 PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT) PLATE 6



Max-Min compaction test

0 blows per each of 1 layers, with 2 psi surcharge lb rammer and

0 inch drop. 11 inch diameter mold

Sample No.	Elev or Depth	Classification	G	LL	PL	% > No. 4	% > 3/4 in.
~~Composite~~ 99805 & 99806		Gravelly Sand(SP)	~~2.63~~	~~NP~~	~~NP~~	~~31~~	~~36~~

Sample No.	99805 & 806			
Natural water content, percent	Unknown			
Optimum water content, percent	7.2			
Max dry density, lb/cu ft	130			

Remarks	Project UPPER SANTA ANA RIVER
Div. No. 99805 & 99806	SEVEN OAKS DAM
	Area Pervious Borrow Materials
	Boring No. TP-3 Date May 1987
	COMPACTION TEST REPORT

ENG FORM 1 JUN 65 2091 PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT) PLATE 7

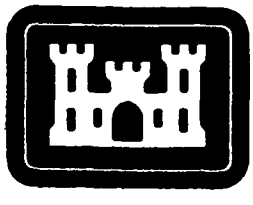
Division Laboratory

Sausalito, California

REPORT
OF
SOIL TESTS

SEVEN OAKS DAM - PROGRAM 3

October 1987



US Army Corps
of Engineers
South Pacific Division

U.S ARMY ENGINEER DIVISION LABORATORY - - SOUTH PACIFIC DIVISION

SOIL TEST RESULT SUMMARY

PROJECT: Upper Santa Ana River - Seven Oaks Dam (Mill Creek core borrow area: Program 3) DATE: OCTOBER 1987

Division Number	Hole Number	Field Sample No.	Depth or Elevation From	To	Laboratory Descriptive Classification	Mechanical Analysis - % Finer: 6	3	2	1	Gravel 3/4	1/2	3/8	#4	Sand #10	#40	#60	#100	Fines #200	Liquid Limit	Plasticity Index	Field Moist. %
102028	CBT 86-17	2	1.0	2.0	Clayey Sand (SC-SM)	-	100	99	99	98	97	97	95	90	61	49	37	26	24	6	
102029	CBT 86-27	5	7.5	10.0	Silty Sand (SM)							100	99	93	54	43	35	25		NP	
102030	CBT 86-29	4	7.5	11.0	Clayey Sand (SC)							100	99	82	45	38	31	23	26	7	
102031	CBT 86-34	6	14.0	16.5	Clayey Sand (SC)							100	99	95	67	58	47	35	22	3	
102032	CBH 86-12	8	15.0	17.0	Silty Gravelly Sand (SW-SM)	-	100	96	96	93	90	86	82	74	62	33	25	18	11		NP	
102033	CBH 86-14	12	26.5	29.5	Clayey Sand (SC-SM)					100	99	99	98	96	80	47	40	33	26	24	6	
102034	CBH 86-16	8	19.0	22.0	Silty Sand (SM)	-	100	99	99	99	98	97	96	93	85	50	38	29	20		NP	
102035	CBH 86-23	15	44.0	47.0	Silty Sand (SM)			-	100	98	97	95	94	89	78	42	33	26	19		NP	
102036	CBH 86-24	6	14.5	16.5	Clayey Sand (SC-SM)			100	98	96	96	95	94	91	76	37	29	23	17	23	5	
102037	CBH 86-31	6	15.0	18.0	Clayey Sand (SC-SM)							100	99	98	88	50	39	31	22	24	6	
102038	CBH 86-32	8	20.0	23.0	Sandy Gravel (GP)	100	90	84	78	72	65	55	46	21	19	11	8	6	4		NP	
102039	CBH 86-33	4	7.0	11.0	Silty Sand (SW-SM)	-	100	98	98	96	95	93	90	82	70	35	24	16	10		NP	
102040 102041	CBH 86-33	6	13.0	15.0	Sandy Gravelly Sand (SW-SM)	100	98	96	93	90	88	85	82	73	59	28	20	14	9		NP	

PLATE I

SPD FORM 66A

U.S ARMY ENGINEER DIVISION LABORATORY - - SOUTH PACIFIC DIVISION

SOIL TEST RESULT SUMMARY

PROJECT: Upper Santa Ana River - Seven Oaks Dam (Mill Creek core borrow area: Program 3) DATE: SEPTEMBER 1987

Division Number	Hole Number	Field Sample No.	Depth or Elevation From	Depth or Elevation To	Laboratory Descriptive Classification	Mechanical Analysis - % Finer: 2	11/2	1	Gravel 3/4	1/2	3/8	#4	Sand #10	#40	#60	#100	Fines #200	Liquid Limit	Plasticity Index	Field Moist. %
102043	CBH 86-38	5	17.0	20.0	Silty Sand (SM)	100	99	99	99	99	98	96	86	45	33	23	15		NP	
102044	CBT 86-8	2	1.5	3.0	Clayey Sand (SC)						100	99	92	61	50	39	28	30	12	
102045	CBT 86-8	3	3.0	5.0	Clayey Sand (SC-SM)						100	99	90	58	46	36	25	26	7	
102046	CBT 86-8	4	5.0	7.0	Silty Sand (SP-SM)						100	96	80	39	28	18	10		NP	
102047	CBT 86-8	5	7.0	10.0	Silty Sand (SP-SM)					100	99	97	84	42	30	20	11		NP	
102049	CBT 86-10	1	0.5	1.5	Clayey Sand (SC)		100	98	96	95	92	85	63	54	46	37	34	17		
102050	CBT 86-10	2	1.5	4.0	Clayey Gravelly Sand (SP-SC)	100	96	93	91	89	83	74	38	26	18	11	29	10		
102051	CBT 86-18	5	5.0	6.0	Clayey Gravelly Sand (SC)			100	99	97	77	66	49	42	33	23	28	13		
102052	CBT 86-18	6	6.0	7.0	Clayey Sand (SC)	100	98	96	96	95	86	74	46	38	31	22	29	14		
102053	CBT 86-19	1	0.0	1.5	Clayey Sand (SC)			100	99	99	96	85	56	47	39	29	24	8		
102054	CBT 86-19	3	4.5	6.0	Clayey Gravelly Sand (SW-SC)	100	95	93	89	85	73	56	29	22	16	11	24	5		
102055	CBT 86-28	3	1.5	4.0	Clayey Sand (SC)		100	99	99	99	98	92	62	52	43	31	28	10		
102056	CBT COMPOSITE	1	0.0	3.0	Clayey Sand (SC)				100	99	98	87	59	50	42	33	35	17		

PLATE 2

SPD FORM 66A

U.S ARMY ENGINEER DIVISION LABORATORY - - SOUTH PACIFIC DIVISION

SOIL TEST RESULT SUMMARY

PROJECT: Upper Santa Ana River - Seven Oaks Dam (Left Abutment Pervious talus material: Program 3) DATE: OCTOBER 1987

Division Number	Hole Number	Field Sample No.	Depth or Elevation From	To	Laboratory Descriptive Classification	Mechanical Analysis - % Finer 3	11/2	Gravel 3/4	1/2	3/8	#4	#10	Sand #40	#60	#100	Fines #200	Liquid Limit	Plasticity Index	Field Moist. %
102057	LAH 86-1	1	0.0	3.0	Silty Sand (SM)		-												
							100	99	98	97	95	79	44	34	26	17		NP	
102058	LAH 86-1	2	3.0	6.0	Silty Sand (SM)		-												
							100	99	99	99	98	85	55	46	37	25		NP	
102059	LAH 86-1	4	7.0	9.0	Silty Sand (SM)														
									100	99	93	75	40	32	24	16		NP	
102060	LAH 86-1	5	9.0	12.0	Silty Sand (SP-SM)		-												
							100	99	98	97	94	78	40	30	21	12		NP	
102061	LAH 86-1	6	12.0	16.0	Silty Gravelly Sand (SW-SM)		100												
							96	96	94	93	85	67	34	26	19	12		NP	
102062	LAH 86-1	7	16.0	17.0	Silty Gravelly Sand (SM)		100												
							96	96	94	92	78	65	35	26	20	13		NP	
102063	LAH 86-1	9	18.0	19.5	Silty Gravelly Sand (SW-SM)	-	96												
						100	93	93	89	88	80	56	25	19	14	9		NP	
102064	LAH 86-1	10	19.5	22.0	Silty Gravelly Sand (SP-SM)		100												
							95	88	85	80	65	56	28	21	15	9		NP	
102065	LAH 86-1	11	22.0	25.0	Silty Gravelly Sand (SP-SM)	-	98												
						100	92	89	85	80	68	52	26	19	13	8		NP	
102066	LAH 86-1	12	26.0	30.0	Silty Gravelly Sand (SW-SM)	-	97												
						100	94	92	89	84	72	59	31	23	17	11		NP	
102067	LAH 86-1	13	30.0	33.0	Silty Gravelly Sand (SM)	-	98												
						100	97	94	89	85	75	68	41	32	24	16		NP	
102068	LAH 86-1	14	33.0	35.0	Silty Gravelly Sand (SM)	100	94												
						97	91	89	85	83	74	63	35	27	20	13		NP	
102069	LAH 86-1	15	35.0	39.0	Silty Gravelly Sand (SW-SM)	-	97												
						100	94	92	88	85	75	59	31	24	17	12		NP	

PLATE 3

SPD FORM 66A

U.S ARMY ENGINEER DIVISION LABORATORY - - SOUTH PACIFIC DIVISION

SOIL TEST RESULT SUMMARY

PROJECT: Upper Santa Ana River - Seven Oaks Dam (Left Abutment Pervious talus material: Program 3) DATE: OCTOBER 1987

Division Number	Hole Number	Field Sample No.	Depth or Elevation From	Depth or Elevation To	Laboratory Descriptive Classification	Mechanical Analysis - % Finer: 3	2	11/2	1	Gravel 3/4	1/2	3/8	#4	Sand #10	#40	#60	#100	Fines #200	Liquid Limit	Plasticity Index	Field Moist. %
102070	LAH 86-1	16	39.0	41.0	Silty Gravelly Sand (SW-SM)	100	92	86	85	83	79	76	66	52	27	21	16	10		NP	
102071	LAH 86-1	17	43.0	44.0	Silty Gravelly Sand (SM)	100	94	90	88	85	82	78	69	58	33	26	20	14		NP	
102072	LAH 86-1	18	44.0	48.0	Silty Gravelly Sand (SP-SM)	100	86	86	82	79	76	73	64	51	26	19	14	9		NP	
102073	LAH 86-1	19	48.0	51.0	Silty Gravelly Sand (SP-SM)	100	82	80	77	75	72	70	61	51	26	20	15	10		NP	
102074	LAH 86-1	20	51.0	54.0	Silty Gravelly Sand (SW-SM)	100	92	90	89	86	84	82	73	58	27	21	16	11		NP	
102075	LAH 86-1	21	54.0	57.0	Silty Gravelly Sand (SM)	100	98	98	93	91	88	85	77	67	35	27	20	14		NP	
102076	LAH 86-1	22	57.0	60.0	Silty Gravelly Sand (SP-SM)	100	92	92	88	85	80	77	67	57	31	23	17	11		NP	
102077	LAH 86-1	23	60.0	63.0	Silty Gravelly Sand (SP-SM)	100	92	86	84	82	78	75	67	57	29	22	17	11		NP	
102078	LAH 86-1	24	63.0	65.0	Silty Gravelly Sand (SM)			-	100	98	95	92	82	66	36	28	22	15		NP	
102079	LAH 86-1	25	65.0	66.0	Silty Gravelly Sand (SP-SM)			100	99	97	94	91	80	61	32	25	19	12		NP	
102080	LAH 86-1	26	66.0	69.0	Silty Gravelly Sand (SM)	-	100	97	96	94	92	90	83	73	45	36	29	20		NP	
102081	LAH 86-1	27	69.0	70.0	Silty Gravelly Sand (SM)	-	100	99	95	92	88	85	75	60	32	25	19	13		NP	
102082	LAH 86-1	28	70.0	73.0	Silty Gravelly Sand (SW-SM)	100	81	81	75	72	68	65	58	45	23	18	14	9		NP	

PLATE 4

SPD FORM 66A

U.S ARMY ENGINEER DIVISION LABORATORY - - SOUTH PACIFIC DIVISION

SOIL TEST RESULT SUMMARY

PROJECT: Upper Santa Ana River - Seven Oaks Dam (Left Abutment Pervious talus material: Program 3) DATE: OCTOBER 1987

Division Number	Hole Number	Field Sample No.	Depth or Elevation From	To	Laboratory Descriptive Classification	Mechanical Analysis - % Finer 3	11/2	Gravel			Sand					Fines	Liquid Limit	Plasticity Index	Field Moist. %
						2	1	3/4	1/2	3/8	#4	#10	#40	#60	#100	#200			
102083	LAH 86-1	29	73.0	75.0	Silty Gravelly Sand (SW-SM)	- 100	95 92	90	88	85	74	58	28	21	15	10		NP	
102084	LAH 86-1	30	75.0	78.5	Silty Gravelly Sand (SW-SM)	100 91	87 86	86	83	80	71	60	32	23	17	11		NP	
102085	LAH 86-1	31	78.5	81.0	Silty Gravelly Sand (SP-SM)	100 79	77 73	71	68	65	58	45	21	16	12	8		NP	
102086	LAH 86-1	32	81.0	83.5	Silty Sandy Gravel (GP-GM)	100 88	88 84	81	74	66	47	38	21	16	12	8		NP	
102087	LAH 86-1	33	83.5	86.5	Silty Gravelly Sand (SW-SM)	100 90	83 81	78	75	73	64	51	26	20	15	10		NP	
102088	LAH 86-1	34	86.5	88.0	Silty Gravelly Sand (SM)	100 96	96 94	92	89	87	77	63	34	27	21	14		NP	
102089	LAH 86-1	35	88.0	90.0	Silty Gravelly Sand (SW-SM)	100 81	81 76	74	72	71	65	56	28	21	16	10		NP	
102090	LH 86-2	1	0.0	1.5	Gravelly Silty Sand (SM)	100 95	95 94	Sp.Gr. = 2.74 94	94	93	89	79	43	33	25	16		NP	
102091	LH 86-2	2	1.5	4.0	Gravelly Silty Sand (SM)	100 97	97 96	96	95	95	92	77	41	31	23	15		NP	
102092	LH 86-2	3	4.0	8.0	Silty Sand (SW-SM)			Sp.Gr. = 2.74 100	99	98	96	72	28	20	14	8		NP	
102092A	LH 86-2	4	8.0	12.0	Silty Gravelly Sand (SW-SM)	- 100	96 92	91	90	89	86	73	36	27	19	12		NP	
102093	LH 86-2	5	12.0	16.0	Silty Gravelly Sand (SM)	100 97	97 93	Sp.Gr. = 2.78 93	91	90	85	70	36	28	21	13		NP	

PLATE 5

SPD FORM 66A

U.S ARMY ENGINEER DIVISION LABORATORY - - SOUTH PACIFIC DIVISION

SOIL TEST RESULT SUMMARY

PROJECT: Upper Santa Ana River - Seven Oaks Dam (Left Abutment Pervious talus material: Program 3) DATE: OCTOBER 1987

Division Number	Hole Number	Field Sample No.	Depth or Elevation From	Depth or Elevation To	Laboratory Descriptive Classification	Mechanical Analysis - % Finer: 6 / 3	2 / 11/2	1 / 3/4	Gravel 1/2	Gravel 3/8	Sand #4	Sand #10	Sand #40	Sand #60	Sand #100	Fines #200	Liquid Limit	Plasticity Index	Field Moist. %
102094	LAH 86-2	6	16.0	20.0	Silty Sand (SM)			— / 100	98	96	87	73	39	29	22	14		NP	
102095	LAH 86-2	7	20.0	25.5	Silty Gravelly Sand (SP-SM)		100 / 93	89 / 86	Sp.Gr. = 2.77 / 83	79	72	62	35	27	20	12		NP	
102096	LAH 86-2	8	25.5	29.0	Silty Gravelly Sand (SP-SM)	- / 100	88 / 88	83 / 80	75	72	64	53	31	24	19	12		NP	
102097	LAH 86-2	9	29.0	32.0	Silty Gravelly Sand (SW-SM)		100 / 97	90 / 88	Sp.Gr. = 2.74 / 82	78	68	55	33	27	21	14		NP	
102098	LAH 86-2	10	32.0	36.0	Silty Gravelly Sand (SW-SM)	- / 100	93 / 93	86 / 85	83	78	68	58	31	23	17	11		NP	
102099	LAH 86-2	11	36.0	39.0	Silty Gravelly Sand (SM)		100 / 97	93 / 93	Sp.Gr. = 2.72 / 92	91	81	73	45	36	28	20		NP	
102100	LAH 86-2	12	41.0	44.0	Silty Gravelly Sand (SM)		100 / 99	94 / 92	90	86	75	66	38	29	22	15		NP	
102101	LAH 86-2	13	44.0	46.5	Gravelly Silty Sand (SM)		100 / 95	94 / 92	Sp.Gr. = 2.73 / 91	89	85	68	37	29	23	17		NP	
102102	LAH 86-2	14	46.5	47.5	Silty Gravelly Sand (SM)	100 / 89	89 / 87	86 / 82	82	80	74	67	40	32	25	17		NP	
102103	LAH 86-3	1	0.0	3.0	Silty Gravelly Sand (SM)		100 / 97	90 / 88	84	80	69	57	37	33	28	19		NP	
102104	LAH 86-3	2	5.0	9.0	Silty Gravelly Sand (SM)		- / 100	99 / 97	95	92	82	59	31	25	20	13		NP	
102105	LAH 86-3	3	9.0	12.0	Silty Sand (SM)		100 / 98	98 / 98	97	96	90	72	38	30	23	15		NP	

PLATE 6

SPD FORM 66A

U.S ARMY ENGINEER DIVISION LABORATORY - - SOUTH PACIFIC DIVISION

SOIL TEST RESULT SUMMARY

PROJECT: Upper Santa Ana River - Seven Oaks Dam (Left Abutment Pervious talus material: Program 3) DATE: OCTOBER 1987

Division Number	Hole Number	Field Sample No.	Depth or Elevation From	To	Laboratory Descriptive Classification	Mechanical Analysis - % Finer: 6 3	2 11/2	1 3/4	Gravel 1/2	3/8	#4	#10	Sand #40	#60	#100	Fines #200	Liquid Limit	Plasticity Index	Field Moist. %
102106	LH 86-3	4	12.0	16.0	Silty Sand (SM)		100 98	98 98	98	98	95	79	43	35	28	19		NP	
1020107	LH 86-3	5	16.0	18.0	Silty Gravelly Sand (SW-SM)		- 100	98 96	95	93	85	64	27	20	14	9		NP	
102108	LH 86-3	6	19.0	22.0	Silty Gravelly Sand (SM)			100 91	89	88	82	68	35	28	21	13		NP	
102109	LH 86-3	7	22.0	25.0	Silty Gravelly Sand (SW-SM)	- 100	97 91	87 83	79	76	67	54	25	19	13	8		NP	
102110	LH 86-3	8	25.0	28.0	Silty Gravelly Sand (SW-SM)		100 98	90 87	83	79	69	50	22	17	13	8		NP	
102111	LH 86-3	9	28.0	31.0	Silty Gravelly Sand (SW-SM)	- 100	96 96	88 81	77	75	66	48	22	17	13	8		NP	
102112	LH 86-3	10	31.0	35.0	Silty Gravelly Sand (SP-SM)	- 100	90 90	87 82	77	74	62	48	21	16	11	7		NP	
102113	LH 86-3	11	35.0	38.5	Silty Gravelly Sand (SW-SM)		100 94	89 86	82	80	71	60	29	21	16	10		NP	
102114	LH 86-3	12	38.5	40.0	Silty Gravelly Sand (SW-SM)	- 100	86 83	82 79	75	72	62	48	23	17	13	9		NP	
102115	LH 86-3	13	40.0	43.0	Silty Gravelly Sand (SW-SM)		- 100	96 93	90	87	76	57	26	20	15	9		NP	
102116	LH 86-3	14	43.0	44.0	Silty Gravelly Sand (SM)		100 98	95 92	88	84	74	63	35	28	21	14		NP	
102117	LAH 86-4	1	0.0	3.0	Silty Gravelly Sand (SM)		100 98	93 92	90	87	78	62	36	30	25	17		NP	

PLATE 7

SPD FORM 66A

U.S ARMY ENGINEER DIVISION LABORATORY - - SOUTH PACIFIC DIVISION

SOIL TEST RESULT SUMMARY

PROJECT: Upper Santa Ana River - Seven Oaks Dam (Left Abutment Pervious talus material: Program 3) DATE: OCTOBER 1987

Division Number	Hole Number	Field Sample No.	Depth or Elevation From	Depth or Elevation To	Laboratory Descriptive Classification	Mechanical Analysis - % Finer: 3	2	11/2	1	Gravel 3/4	1/2	3/8	#4	Sand #10	#40	#60	#100	Fines #200	Liquid Limit	Plasticity Index	Field Moist. %
102118	LAH 86-4	2	3.0	5.0	Silty Gravelly Sand (SM)			100	97	96	93	89	79	62	39	34	29	19		NP	
102119	LAH 86-4	3	5.0	8.0	Silty Gravelly Sand (SM)	100	94	84	83	81	78	76	67	53	33	28	24	16		NP	
102120	LAH 86-4	4	8.0	10.0	Silty Gravelly Sand (SM)	100	87	87	86	83	81	79	71	59	37	32	26	18		NP	
102121	LAH 86-4	5	10.0	12.0	Gravelly Silty Sand (SM)	-	100	95	93	92	91	89	84	70	39	31	24	16		NP	
102122	LAH 86-4	6	12.0	15.0	Silty Sand (SM)	100	96	96	96	95	93	92	86	69	36	28	21	13		NP	
102123	LAH 86-4	7	15.0	17.0	Silty Gravelly Sand (SM)	100	94	89	88	86	84	82	77	64	34	26	20	13		NP	
102124	LAH 86-4	8	17.0	19.0	Silty Gravelly Sand (SM)	-	100	97	93	90	89	87	81	68	37	29	22	15		NP	
102125	LAH 86-4	9	19.0	20.0	Silty Gravelly Sand (SW-SM)			100	99	97	93	91	82	63	30	23	17	11		NP	
102126	LAH 86-4	10	20.0	22.0	Silty Gravelly Sand (SW-SM)			100	96	90	84	79	67	51	25	19	14	9		NP	
102127	LAH 86-4	11	22.0	25.0	Silty Gravelly Sand (SW-SM)	100	89	86	83	79	75	73	62	43	19	15	11	7		NP	
102128	LAH 86-4	12	25.0	26.0	Silty Gravelly Sand (SM)	-	100	98	95	94	88	86	77	61	32	26	21	14		NP	
102129	LAH 86-4	13	26.0	27.0	Silty Gravelly Sand (SW-SM)	100	91	86	82	78	74	73	63	51	25	19	14	9		NP	
102130	LAH 86-4	14	27.0	28.0	Silty Gravelly Sand (SW-SM)	100	96	92	84	82	77	73	65	48	20	15	10	6		NP	

PLATE 8

SPD FORM 66A

U.S ARMY ENGINEER DIVISION LABORATORY - - SOUTH PACIFIC DIVISION

SOIL TEST RESULT SUMMARY

PROJECT: Upper Santa Ana River - Seven Oaks Dam (Left Abutment Pervious talus material: Program 3) DATE: OCTOBER 1987

Division Number	Hole Number	Field Sample No.	Depth or Elevation From	To	Laboratory Descriptive Classification	Mechanical Analysis - % Finer												Liquid Limit	Plasticity Index	Field Moist. %
						3	11/2	Gravel			Sand						Fines			
						2	1	3/4	1/2	3/8	#4	#10	#40	#60	#100	#200				
102131	LAH 86-4	15	28.0	31.0	Silty Gravelly Sand (SW-SM)	- 100	97 94	92	89	85	77	59	28	22	17	11		NP		
102132	LAH 86-4	16	31.0	33.0	Silty Sand (SM)		100 98	96	94	92	86	73	41	33	26	18		NP		
102133	LAH 86-4	17	33.0	35.0	Silty Gravelly Sand (SW-SM)	100 97	89 86	84	81	78	69	60	32	25	19	12		NP		
102134	LAH 86-5	1	0.0	3.0	Silty Gravelly Sand (SW-SM)	100 95	95 90	88	85	82	74	58	26	19	13	7		NP		
102135	LAH 86-5	2	3.0	6.0	Silty Gravelly Sand (SM)	- 100	96 91	88	84	82	74	60	33	26	20	13		NP		
102136	LAH 86-5	3	6.0	7.0	Silty Gravelly Sand (SW-SM)	- 100	98 96	94	91	88	80	66	33	25	19	12		NP		
102137	LAH 86-5	4	7.0	10.0	Silty Gravelly Sand (SM)		100 95	93	90	88	81	68	37	29	22	15		NP		
102138	LAH 86-5	5	10.0	12.0	Silty Gravelly Sand (SW-SM)	100 88	85 84	83	80	78	71	60	32	25	19	12		NP		
102139	LAH 86-5	7	15.0	17.0	Silty Gravelly Sand (SW-SM)	- 100	97 95	92	88	85	75	62	32	24	18	11		NP		
102140	LAH 86-5	8	17.0	19.0	Silty Gravelly Sand (SW-SM)	100 94	94 88	87	84	81	73	61	32	25	19	12		NP		

PLATE 9

SPD FORM 66A

Division Laboratory

Sausalito, California

REPORT
OF
SOIL TESTS

SEVEN OAKS DAM (UPPER SANTA ANA RIVER DAM)
IMPERVIOUS CORE

January 1988

US Army Corps
of Engineers
South Pacific Division

REPORT
OF
SOIL TESTS

SEVEN OAKS DAM (UPPER SANTA ANA RIVER DAM)
IMPERVIOUS CORE

January 1988

AUTHORIZATION

1. Testing services were authorized by the Los Angeles District per DA Form 2544 No. E86 87 0085 dated 21 April 1987.

SAMPLES

2. On 3 & 10 February 1987, twenty barrels and seven sacks of samples were received at the Laboratory. Identification of the samples is shown on the Soil Test Result Summary plates.

TESTING PROGRAM

3. The program was in accordance with the test request as per Detailed Laboratory Testing Program.

TESTS AND TEST METHODS

4. Grain-size Analysis, Atterberg Limits, SCS Dispersion, Specific Gravity, Triaxial Compression, Compaction, Consolidation and Permeability. Testing conformed to the procedure described in Engineer Manual (EM) 1110-2-1906, "Laboratory Soil Testing," 30 November 1970.

CLASSIFICATION

5. The soil was classified in accordance with "The Unified Soil Classification System," Technical Manual (TM) 3-357, Appendix A, April 1960.

RESULTS

6. Results of tests are shown on the following plates:

Subject	Plate No.
(Phase I)	
Soil Test Result Summary	1-4
Gradation Curves	5-13

(Phase II)

Soil Test Result Summary	14-15
Gradation Curves	16-20
Specific Gravity	21
Permeability	22
Triaxial Compression Test Report "R" Tests	23-26
Compaction Test Report	27-32
Consolidation	33-44

(Phase III)

Soil Test Result Summary	45-46
Gradation Curves	47-51
Specific Gravity	52
Permeability	53
Triaxial Compression Test Report "R" Tests	54-55
Compaction Test Report	56-61
Consolidation Test Report	62-73

(Phase IV)

Soil Test Result Summary	74
Gradation Curves	75
Permeability	76-78
Triaxial Compression Test Report	79-87
Compaction Test Report	88
Consolidation Test Report	89-124

U.S ARMY ENGINEER DIVISION LABORATORY - - SOUTH PACIFIC DIVISION

SOIL TEST RESULT SUMMARY

PROJECT: Seven Oaks Dam - Mill Creek Impervious Core (Phase I) DATE: MARCH 1987

Division Number	Hole Number	Field Sample No.	Depth or Elevation From	Depth or Elevation To	Laboratory Descriptive Classification	Mechanical Analysis - % Finer: 3	2	11/2	1	Gravel 3/4	1/2	3/8	#4	#10	Sand #40	#60	#100	Fines #200	Liquid Limit	Plasticity Index	Field Moist. %
100029	CBT 86-30	1	0.75	5.0	Silty Clayey Sand(SC)							100	99	93	65	57	50	41	33	11	3.0
100030	CBT 86-30	2	5.0	11.0	Clayey Sand(SC)	-	100	99	99	99	99	98	97	85	48	36	26	17	33	13	0.9
100031	CBT 86-31	3	1.0	2.5	Clayey Sand(SC)								100	97	76	65	56	45	38	19	3.6
100032	CBT 86-31	4	11.0	17.0	Silty Sand (SW-SM)			-	100	99	98	98	95	83	44	32	22	12		NP	1.9
100033	CBT 86-32	5	1.0	3.0	Clayey Sand(SC)							100	99	91	61	51	41	32	34	15	3.5
100034	CBT 86-32	6	3.0	5.0	Silty Clayey Sandy(SC)						100	99	99	92	63	54	45	35	32	10	3.0

PLATE I, PHASE I

SPD FORM 66A

U.S ARMY ENGINEER DIVISION LABORATORY - - SOUTH PACIFIC DIVISION

SOIL TEST RESULT SUMMARY

PROJECT: Seven Oaks Dam - Mill Creek Impervious Core (Phase I) DATE: MARCH 1987

Division Number	Hole Number	Field Sample No.	Depth or Elevation From	Depth or Elevation To	Laboratory Descriptive Classification	Mechanical Analysis - % Finer: 6	3	2	11/2	1	3/4	1/2	3/8	#4	#10	#40	#60	#100	#200	Liquid Limit	Plasticity Index	Field Moist. %
100035	CBH 86-29	7	0	+10	Clayey Sand (SC-SM)					100	99	98	98	95	88	59	45	35	28	23	7	2.0
100036	CBH 86-29	8	0	+10	Silty Sand(SM)					-	100	99	99	97	89	66	57	49	37	20	3	1.5
100037	CBH 86-11	9	1.0	4.0	Clayey Sand (SC)	100	99	99	99	99	98	98	98	96	89	65	57	50	40	37	17	2.0
100038	CBH 86-11	10	1.0	4.0	Clayey Sand (SC)					100	99	99	99	97	87	58	49	42	33	33	14	2.6
100039	CBH 86-10	11	0.0	6.5	Clayey Sand (SC)					100	99	99	99	96	88	55	43	33	25	31	13	2.4
100040	CBH 86-10	12	0.0	6.5	Clayey Gravelly Sand (SC)	-	100	99	97	95	92	91	90	78	65	37	28	20	13	27	9	4.1

Comment: These moisture contents were from Air Drying.

PLATE 2, PHASE I

SPD FORM 66A

U.S ARMY ENGINEER DIVISION LABORATORY - - SOUTH PACIFIC DIVISION

SOIL TEST RESULT SUMMARY

PROJECT: Seven Oaks Dam - Mill Creek Impervious Core (Phase I) DATE: MARCH 1987

Division Number	Hole Number	Field Sample No.	Depth or Elevation From	Depth or Elevation To	Laboratory Descriptive Classification	Mechanical Analysis - % Finer Gravel 6 3	Gravel 2 11/2	Gravel 1 3/4	Gravel 1/2	Gravel 3/8	Sand #4	Sand #10	Sand #40	Sand #60	Sand #100	Fines #200	Liquid Limit	Plasticity Index	Field Moist. %
100041	CBH 86-17	13	0.0	6.0	Clayey Sand (SC-SM)			100	99	99	98	93	62	50	39	28	28	6	5.2
100042	CBH 86-17	14	0.0	6.0	Silty Sand (SW-SM)			100 99	99	99	96	86	44	30	19	10		NP	3.8
100043	CBT 86-33	15	1.5	5.0	Clayey Sand (SC-SM)						100	97	75	68	59	47	23	6	6.5
100044	CBT 86-33	16	5.0	8.0	Clayey Sand (SC)					100	99	95	64	53	41	29	26	7	7.0
100045	CBT 86-34	17	0.75	5.0	Clayey Sand (SC)						100	96	76	68	61	50	25	8	9.8
100046	CBT 86-34	18	0.75	5.0	Sandy Clay (CL)						100	97	78	70	63	52	27	11	10.9
100047	CBT 86-34	19	27.5	32.0	Sandy Clay (CL)						100	97	82	76	70	59	27	11	12.1
100048	CBH 86-35	20	9.5	14.0	Silty Sand (SM)		- 100	99 99	98	97	93	84	56	45	34	23		* NP	7.7
100049	CBH 86-36	21	14.0	19.0	Silty Sand (SM)	- 100	99 99	98 97	96	95	91	81	46	35	25	15		* NP	6.5

PLATE 3, PHASE I

SPD FORM 66A

U.S ARMY ENGINEER DIVISION LABORATORY - - SOUTH PACIFIC DIVISION

SOIL TEST RESULT SUMMARY

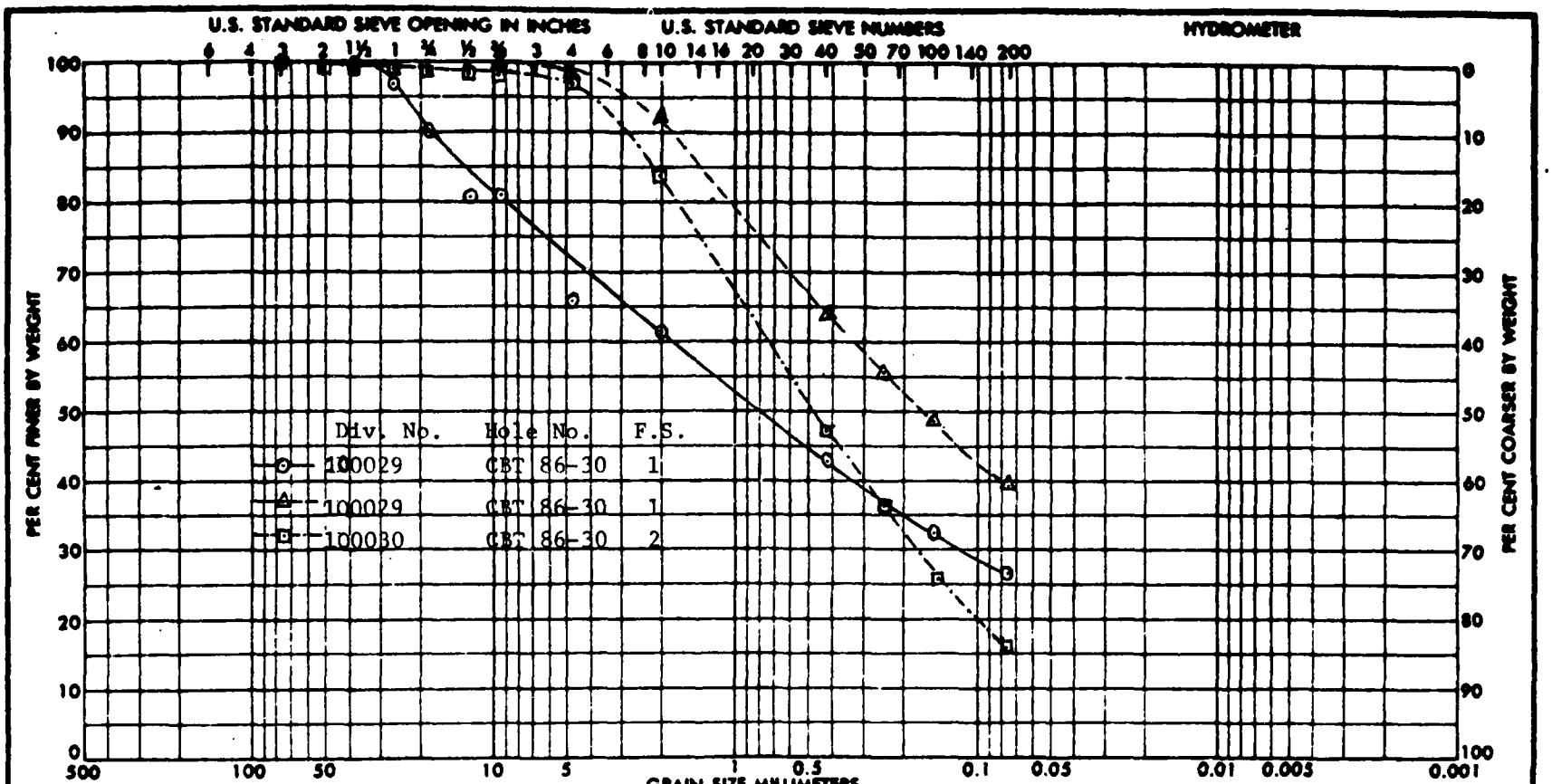
PROJECT: Seven Oaks Dam - Mill Creek Impervious Core (Phase I) DATE: MARCH 1987

Division Number	Hole Number	Field Sample No.	Depth or Elevation From	Depth or Elevation To	Laboratory Descriptive Classification	Mechanical Analysis - % Finer: 6	3	2	1 1/2	3/4	3/8	#4	#10	#40	#60	#100	#200	Liquid Limit	Plasticity Index	Field Moist. %
100050	CBH 86-38	22	7.0	12.0	Silty Sand (SM)	100	99	99	99	98	97	95	82	43	30	22	13		NP	5.6

Comment: * NP but contains some plastic fines-not enough for rollout.

PLATE 4, PHASE I

SPD FORM 66A



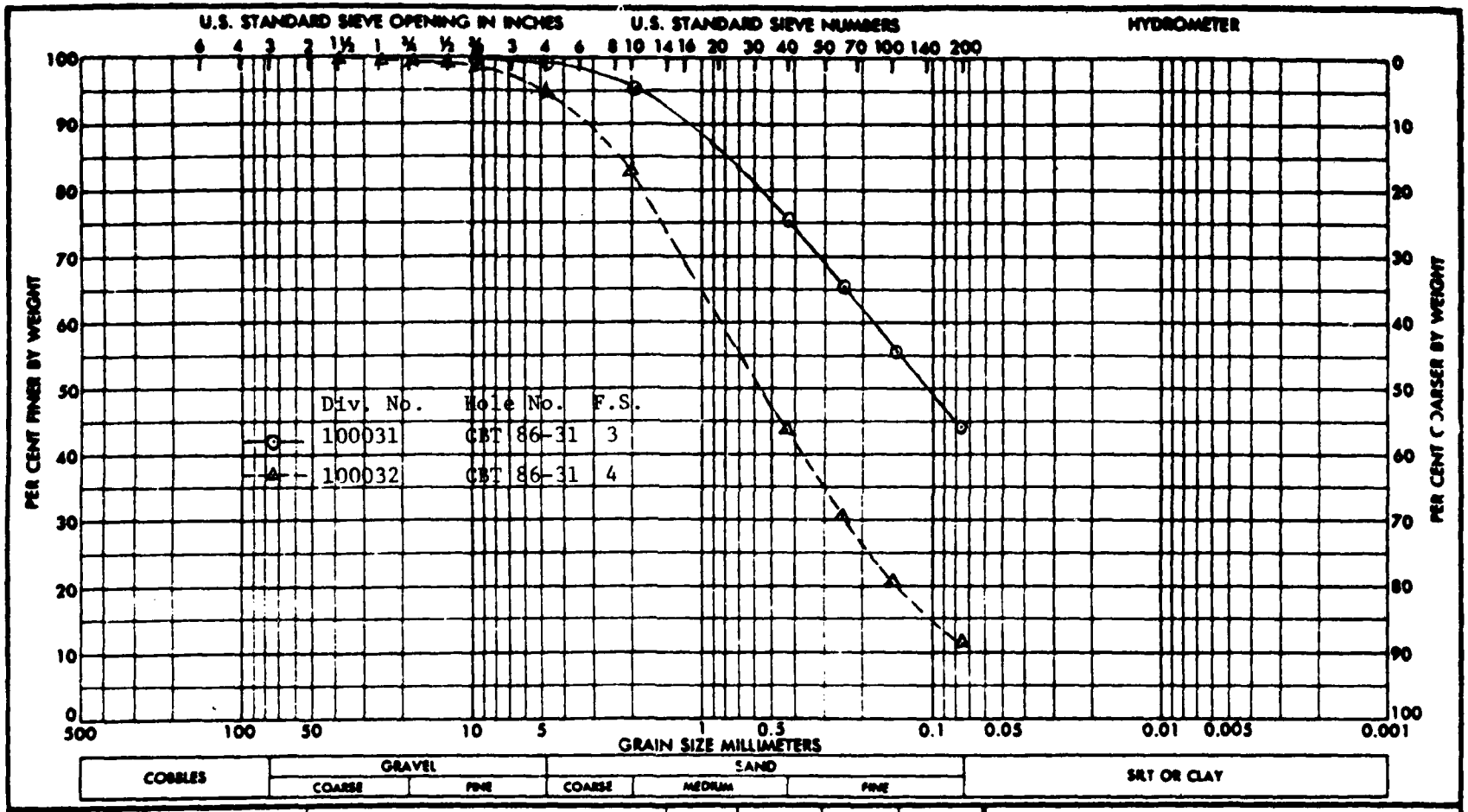
SAMPLE NO.	ELEV OR DEPTH	CLASSIFICATION	NAT W%	LL	PL	PI
—○—	Trommel only	Silty Clayey Sand(SC)	3.3	33	22	11
--△--	Tylab after Trommel	"	3.3	33	22	11
--□--	5.0' - 11.0'	Clayey Sand(SC)	0.9	33	20	13

GRADATION CURVES

PROJECT Upper Santa Ana
Seven Oaks Dam
AREA (PHASE I)
BORING NO. CBT 86-30
DATE March 1987

PLATE 5, PHASE I

ENG FORM 2087 1 MAY 63 REPLACES WES FORM NO. 1241, SEP 1962, WHICH IS OBSOLETE.



SAMPLE NO.	ELEV OR DEPTH	CLASSIFICATION	NAT W%	LL	PL	PI
3	1.0' - 2.5'	Clayey Sand(SC)	3.6	38	19	19
4	11.0' - 17.0	Silty Sand(SW-SM)	1.9			NP

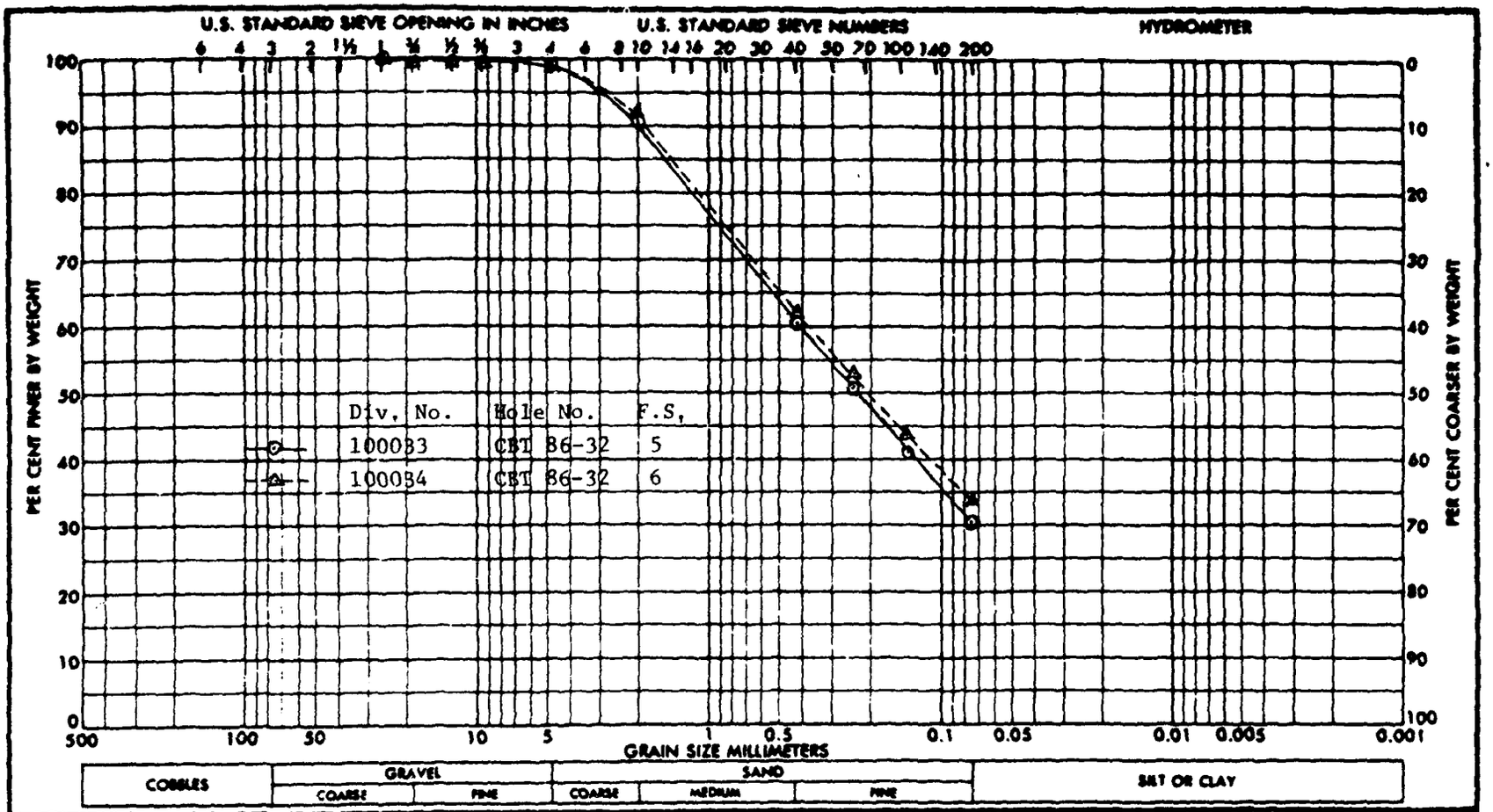
GRADATION CURVES

PROJECT Upper Santa Ana
Seven Oaks Dam
AREA (PHASE I)
BORING NO. CBT 86-31
DATE March 1987

PLATE 6, PHASE I

ENG FORM 2087 1 MAY 63 REPLACES WES FORM NO. 1241, SEP 1962, WHICH IS OBSOLETE.

U.S. GOVERNMENT PRINTING OFFICE 1963 OF—700-100



SAMPLE NO.	ELEV OR DEPTH	CLASSIFICATION	NAT W%	LL	PL	PI
5	1.0' - 3.0'	Clayey Sand(SC)	3.5	34	19	15
6	3.0' - 5.0'	Silty Clayey Sand(SC)	3.0	32	22	10

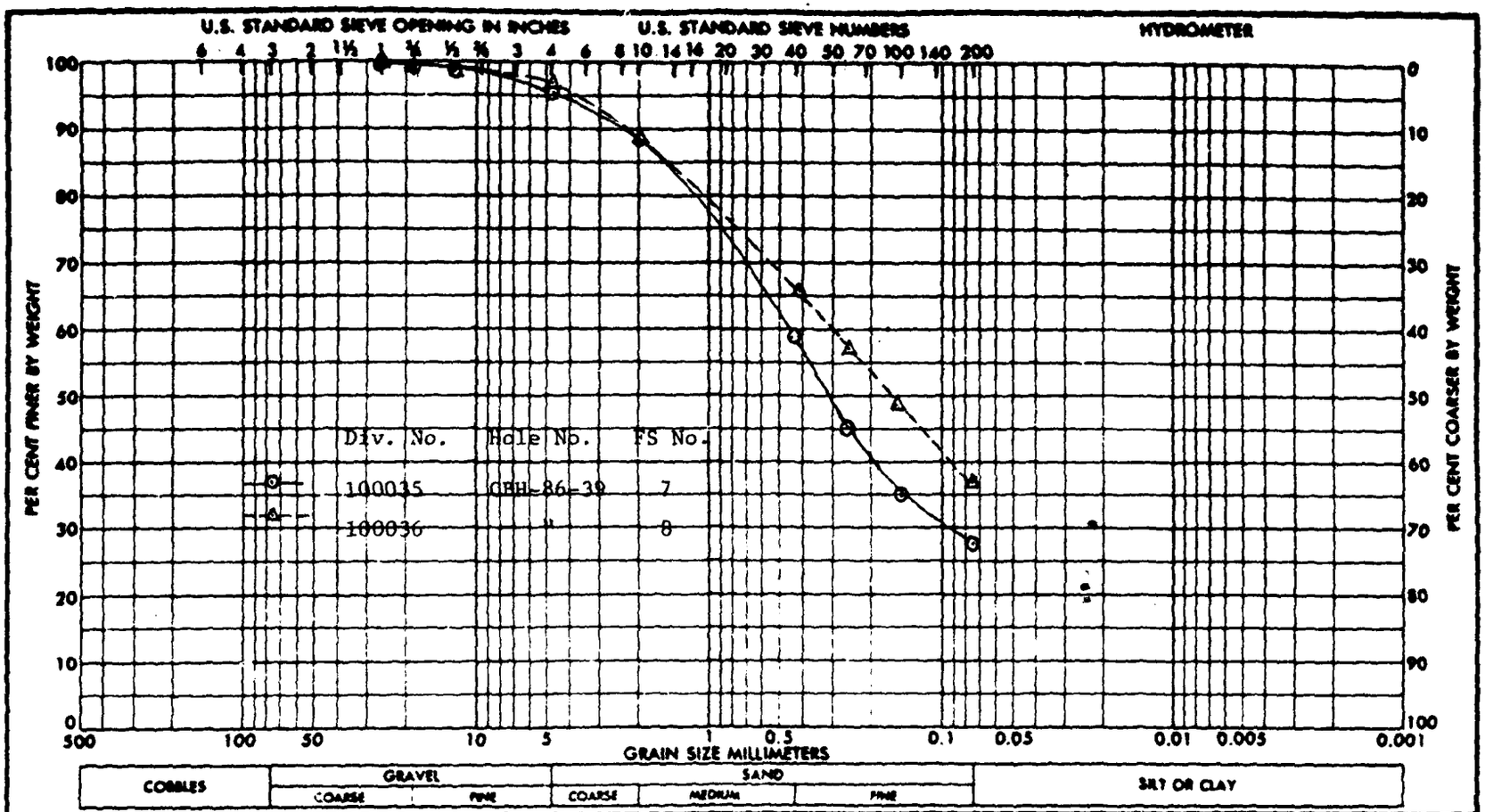
GRADATION CURVES

PROJECT Upper Santa Ana
Seven Oaks Dam
AREA (PHASE I)
BORING NO. CBT 86-32
DATE March 1987

PLATE 7, PHASE I

ENG FORM 2087
1 MAY 63

REPLACES WES FORM NO. 1241, SEP 1962, WHICH IS OBSOLETE.



SAMPLE NO.	ELEV OR DEPTH	CLASSIFICATION	NAT W%	LL	PL	PI
7	0' -+10'	Clayey Sand(SC-SM)	2	23	16	7
8	"	Silty Sand(SM)	15	20	17	3

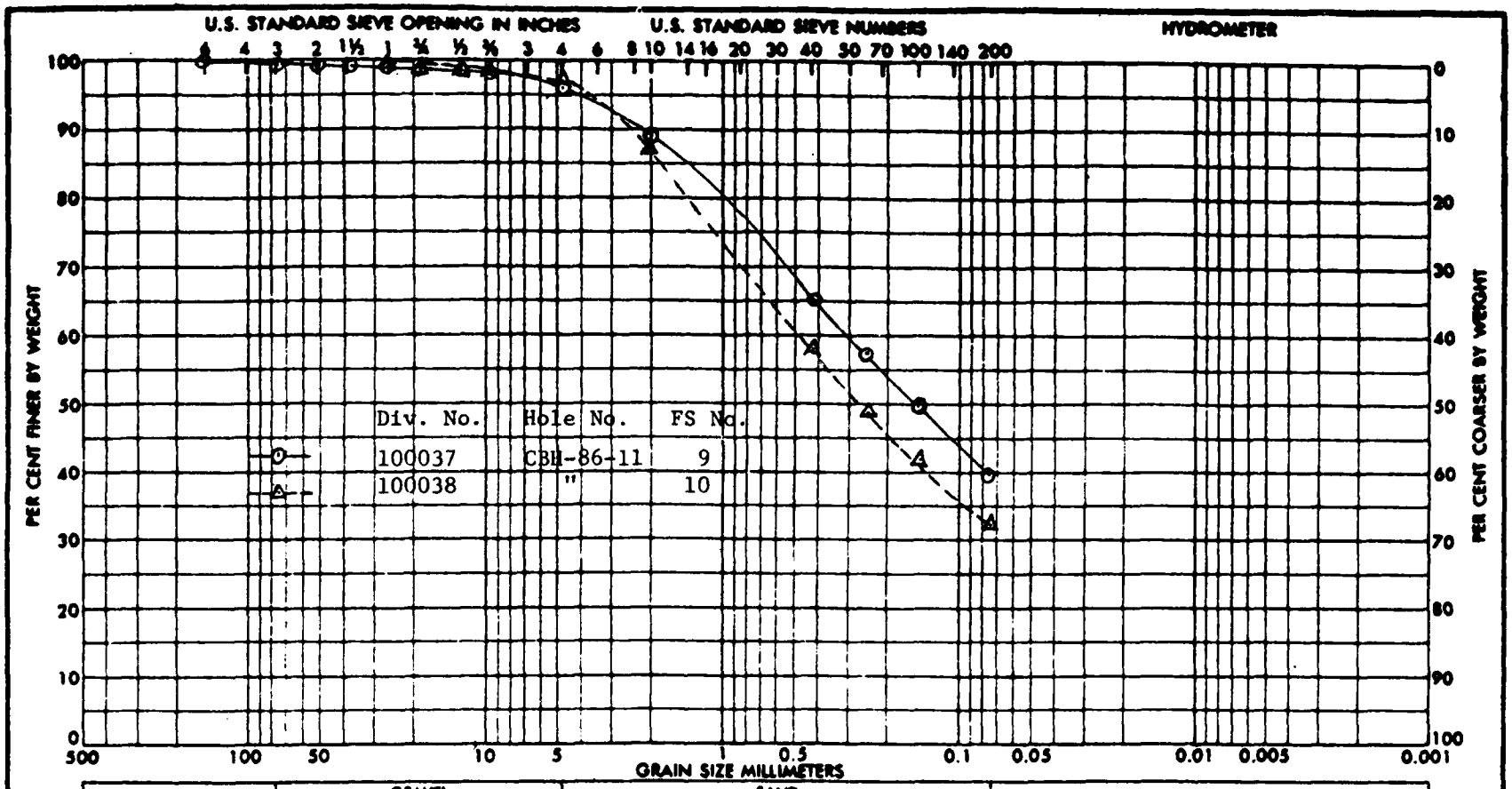
GRADATION CURVES

PROJECT UPPER SANTA ANA RIVER
(PHASE I)
AREA Seven Oaks Dam
BORING NO. CBH 86-39
DATE April 1987

PLATE 8, PHASE I

ENG FORM 2087
1 MAY 63

REPLACES WES FORM NO. 1241, SEP 1962, WHICH IS OBSOLETE.



SAMPLE NO.	ELEV OR DEPTH	CLASSIFICATION	NAT W%	LL	PL	PI
9	1'-4'	Clayey Sand(SC)	2	37	20	17
10	"	Clayey Sand(SC)	2.6	33	19	14

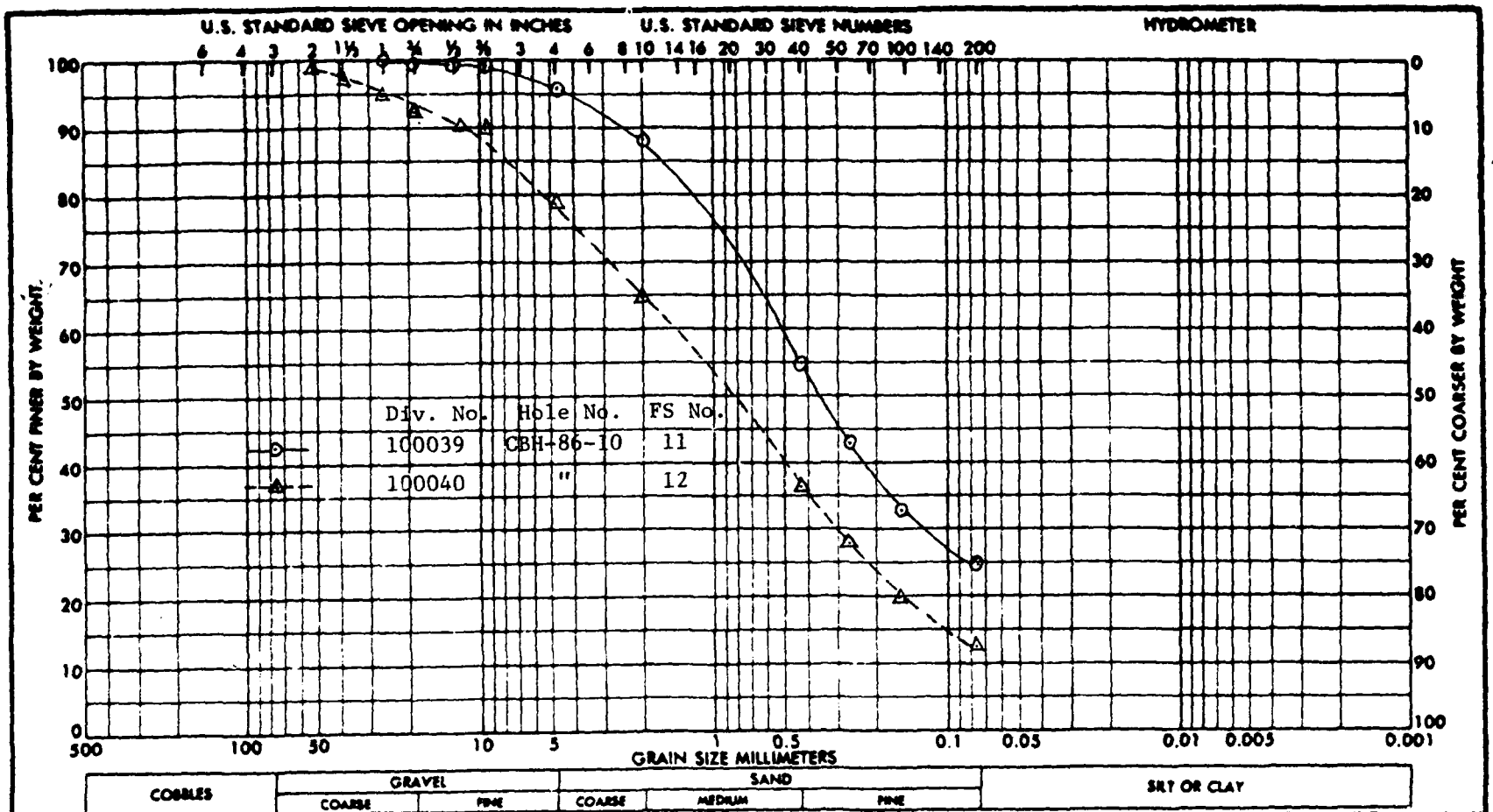
GRADATION CURVES

PROJECT UPPER SANTA ANA RIVER
(PHASE I)
AREA Seven Oaks Dam
BORING NO. CBH 86-11
DATE April 1987

PLATE 9, PHASE I

ENG FORM 2087
1 MAY 63

REPLACES WES FORM NO. 1241, SEP 1962, WHICH IS OBSOLETE.



SAMPLE NO.	ELEV OR DEPTH	CLASSIFICATION	NAT W%	LL	PL	PI
11	0' - 6.5'	Clayey Sand(SC)	2.4	31	18	13
12	" "	Clayey Gravelly Sand(SC)	4.1	27	18	9

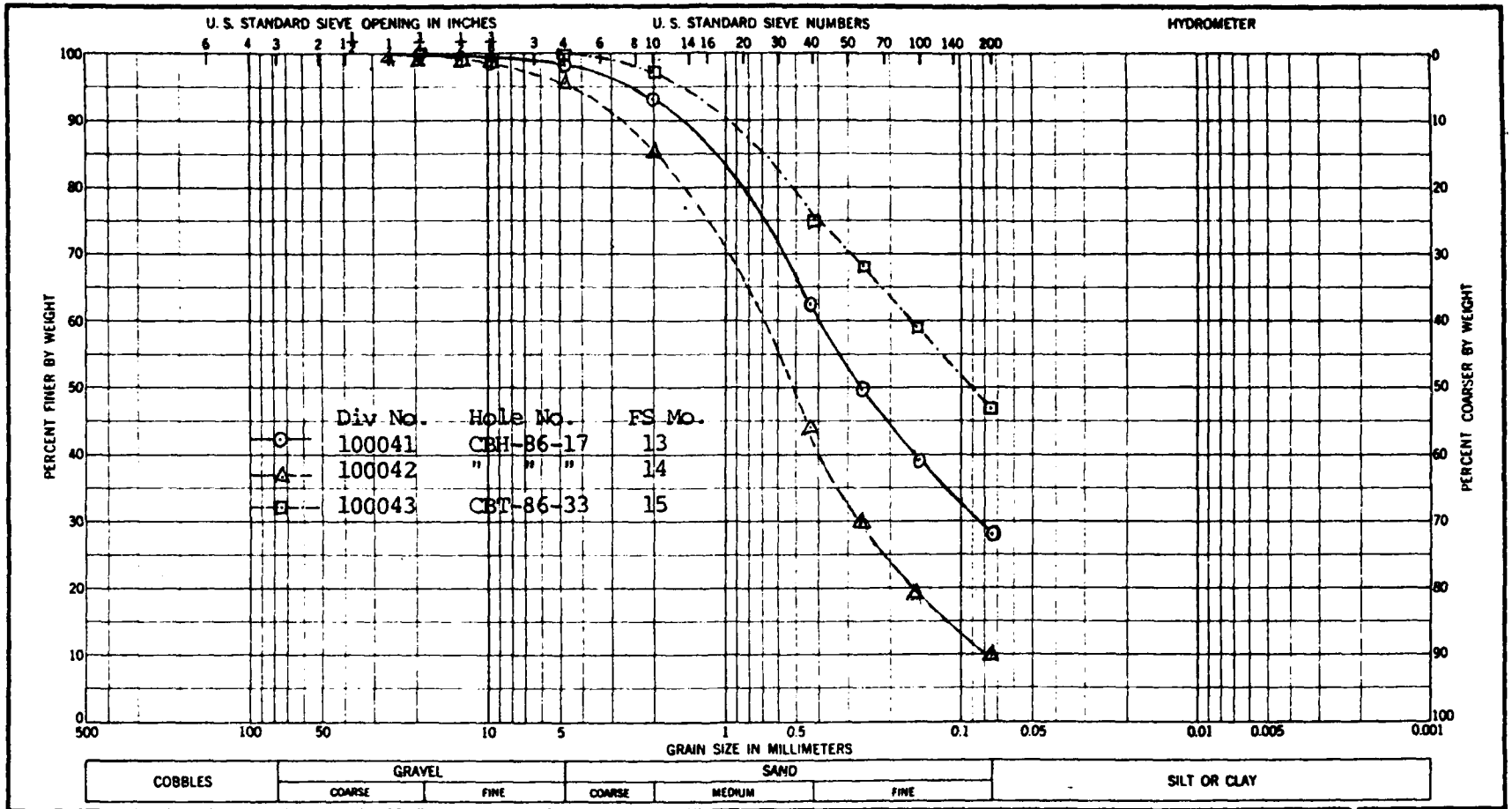
GRADATION CURVES

PROJECT: UPPER SANTA ANA RIVER
(PHASE I)
AREA: Seven Oaks Dam
BORING NO.: CBH 86-10
DATE: April 1987

PLATE 10, PHASE I

ENG FORM 2087
1 MAY 63

REPLACES WES FORM NO. 1241, SEP 1962, WHICH IS OBSOLETE.



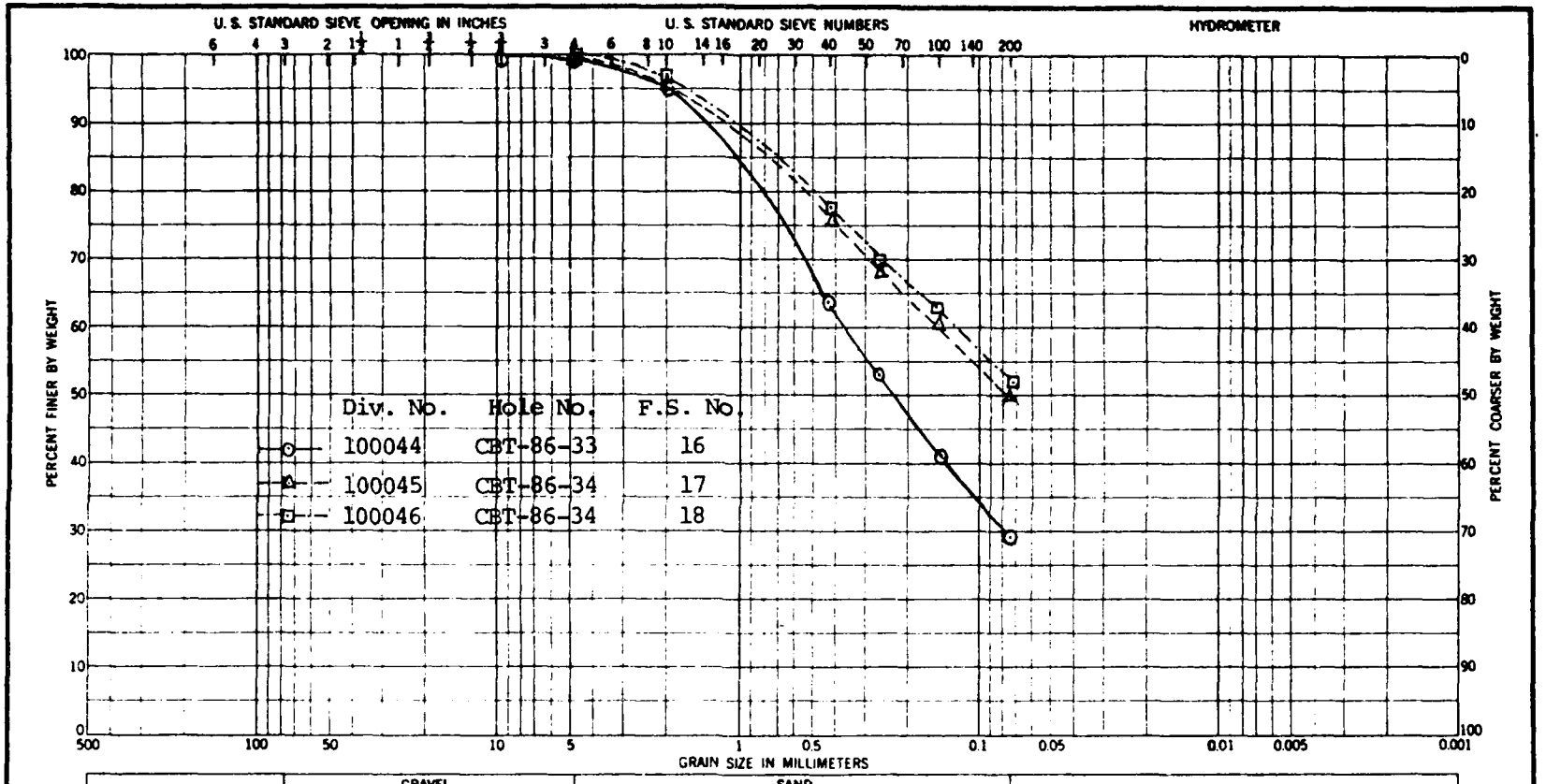
Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
13	0 - 6	Clayey Sand(SC-SM)	5.2	28	22	6
14	0 - 6	Silty Sand (SW - SM)	3.8			NP
15	1.5 - 5	Clayey Sand (SC-SM)	6.5	23	17	6

GRADATION CURVES

Project: SEVEN OAKS DAM
Mill Creek Impervious Core
Area: (Phase I)
Boring No.
Date: July 1987

PLATE 11, PHASE I

ENG FORM 1 MAY 63 2087



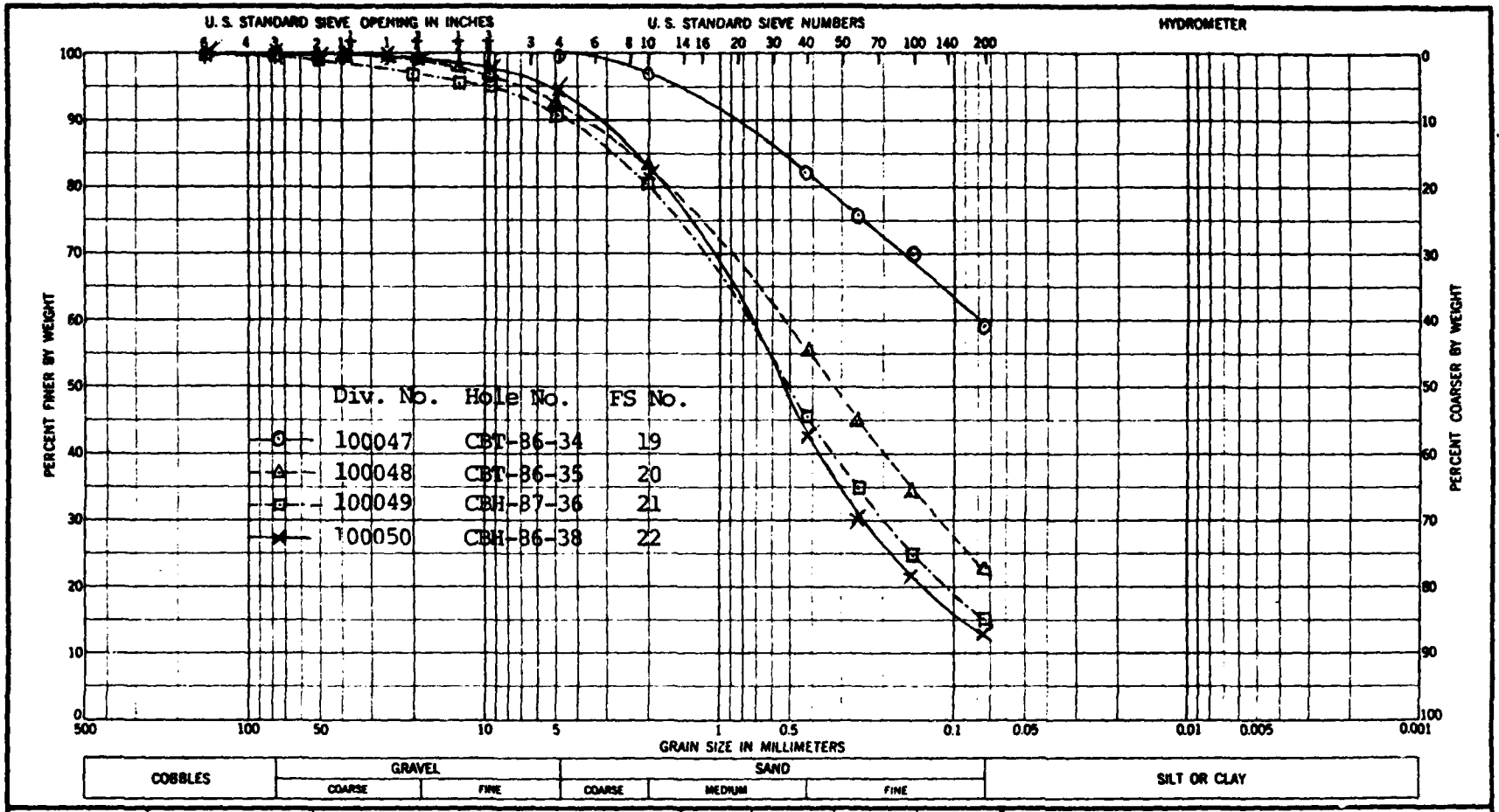
Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
16	5 - 8	Clayey Sand (SC)	7.0	26	19	7
17	0.75 - 5	Clayey Sand (SC)	9.8	25	17	8
18	0.75 - 5	Sandy Clay (CL)	10.9	27	16	11

GRADATION CURVES

Project: SEVEN OAKS DAM
Mill Creek Impervious Core
Area: (Phase I)
Boring No.
Date: July 1987

PLATE 12, PHASE I

ENG FORM 1 MAY 63 2087



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
19	27.5-32.0	Sandy Clay (CL)	12.1	27	16	11
20	9.5-14	Silty Sand (SM)	7.7			NP
21	14-19	Silty Sand (SM)	6.5			NP
22	7-12	Silty Sand (SM)	5.6			NP

GRADATION CURVES

Project: SEVEN OAKS DAM
Mill Creek Impervious Core
Area: (Phase I)
Boring No.
Date: July 1987

PLATE 13, PHASE I

ENG FORM 1 MAY 63 2087

U.S ARMY ENGINEER DIVISION LABORATORY - - SOUTH PACIFIC DIVISION

SOIL TEST RESULT SUMMARY

PROJECT: Upper Santa Ana River - Seven Oaks Dam (Impervious Core: Phase II) DATE: AUGUST 1987

Division Number	Hole Number	Field Sample No.	Depth or Elevation From	Depth or Elevation To	Laboratory Descriptive Classification	Mechanical Analysis - % Finer Gravel 6	3	2	1.5	1	3/4	1/2	3/8	Sand #4	#10	#40	#60	#100	Fines #200	Liquid Limit	Plasticity Index	Field Moist. %
100029	CBT 86-38 Blend 1A	1	0.75	5.0										100	92	64	54	45	34			
100030	CBT 86-30 Blend 2A	2	5.0	11.0				100	99	99	99	99	98	97	91	58	45	35	24			
100045 100046	CBT 86-34 Blend 3A	17 & 18	0.75	5.0										100	97	78	71	63	52			
100047	CBT 86-34 Blend 5A	19	27.5	32.0										100	97	80	73	65	53			
100050	CBT 86-38 Blend 6A	22	7.0	12.0		100	99	99	99	99	98	98	97	95	86	51	39	28	18			

SPD FORM 66A

PLATE 14, PHASE II

U.S ARMY ENGINEER DIVISION LABORATORY - - SOUTH PACIFIC DIVISION

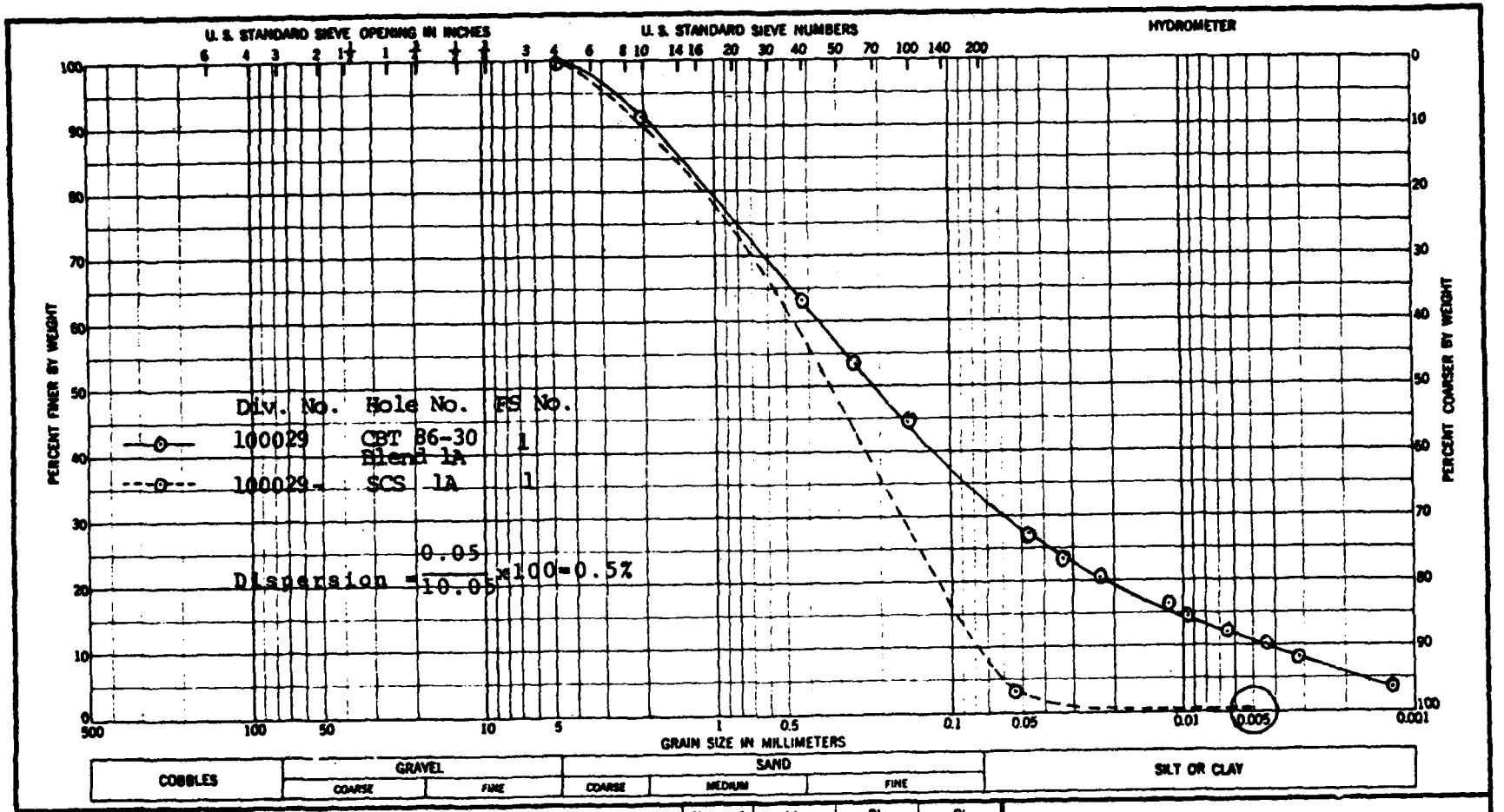
SOIL TEST RESULT SUMMARY

PROJECT: Upper Santa Ana River - Seven Oaks Dam (Impervious Core: Phase II) DATE: AUGUST 1987

Division Number	Hole Number	Field Sample No.	Depth or Elevation From	To	Laboratory Descriptive Classification	Mechanical Analysis - % Finer Gravel 6	3	2	1.5	1	3/4	1/2	3/8	Sand #4	#10	#40	#60	#100	Fines #200	Liquid Limit	Plas-ticity Index	Field Moist. %
100029	SCS 1A	1	0.75	5.0										100								
100030	SCS 2A	2	5.0	11.0				100	99	99	99	99	98	97								
100045 100046	SCS 3A	17 & 18	0.75	5.0										100								
100047	SCS 5A	19	27.5	32.0										100								
100050	SCS 6A	22	7.0	12.0		100	99	99	99	99	98	98	97	95								

SPD FORM 66A

PLATE 15, PHASE II



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
1	0.75 - 5.0					

GRADATION CURVES

Project: UPPER SANTA ANA RIVER
SEVEN OAKS DAM

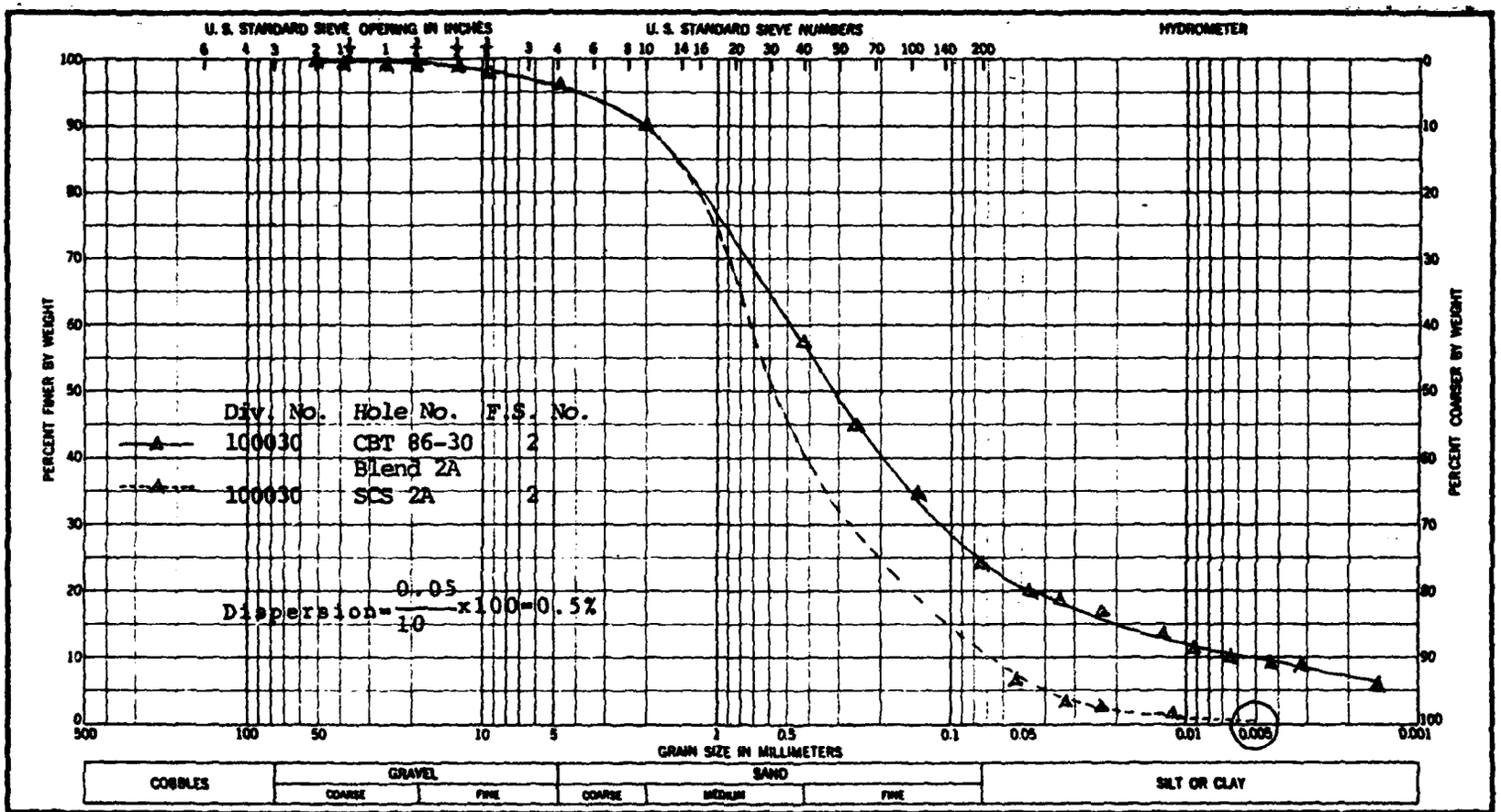
Area: Impervious Core (Phase II)

Boring No.

Date: August 1987

PLATE 16, PHASE II

ENG FORM 1 MAY 63 2087



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
2	5.0 - 11.0					

GRADATION CURVES

Project: UPPER SANTA ANA RIVER
SEVEN OAKS DAM
Impervious Core (Phase II)

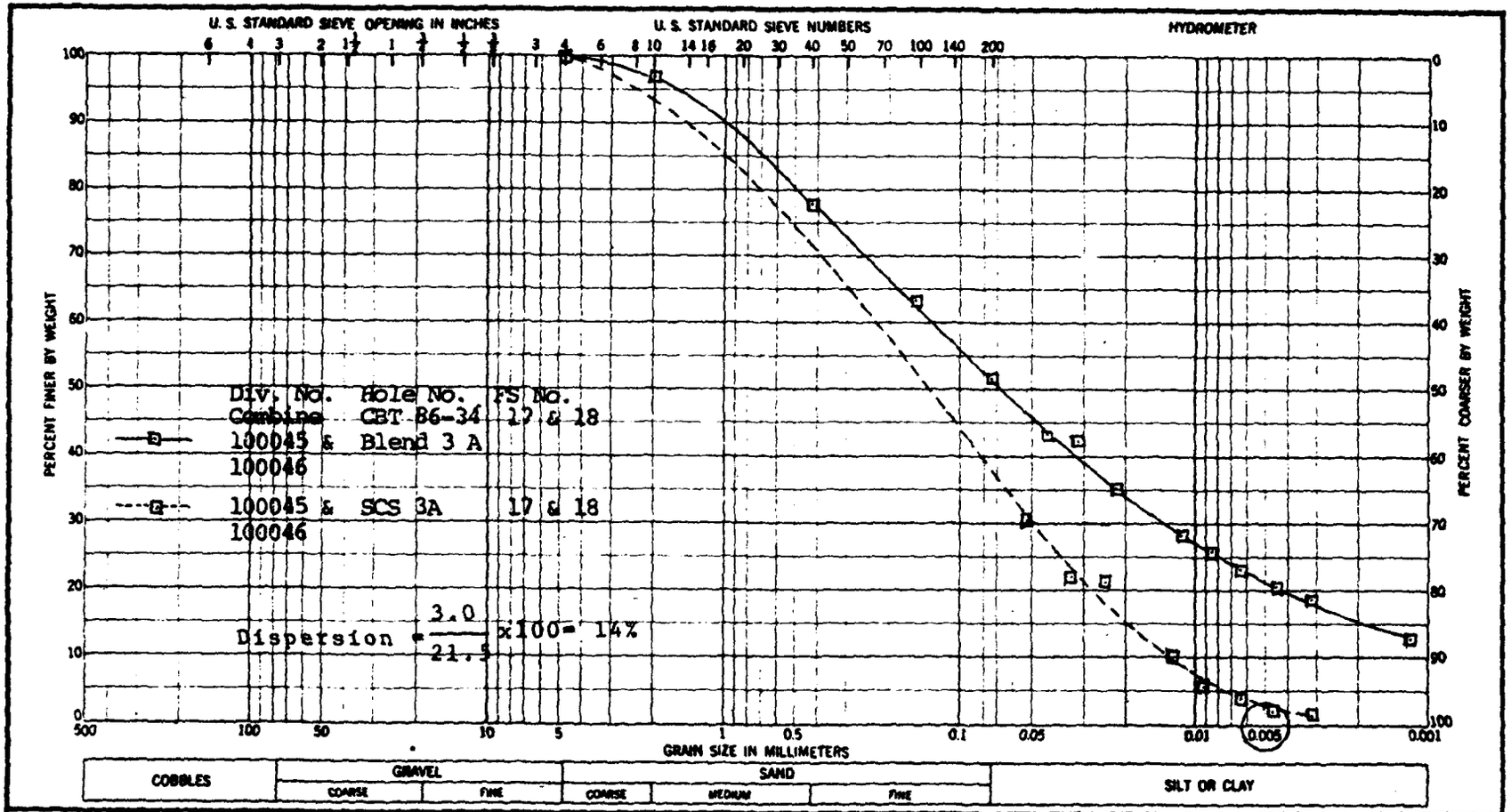
Area:

Boring No.:

Date: August 1987

PLATE 17, PHASE II

ENG FORM 1 MAY 63 2087



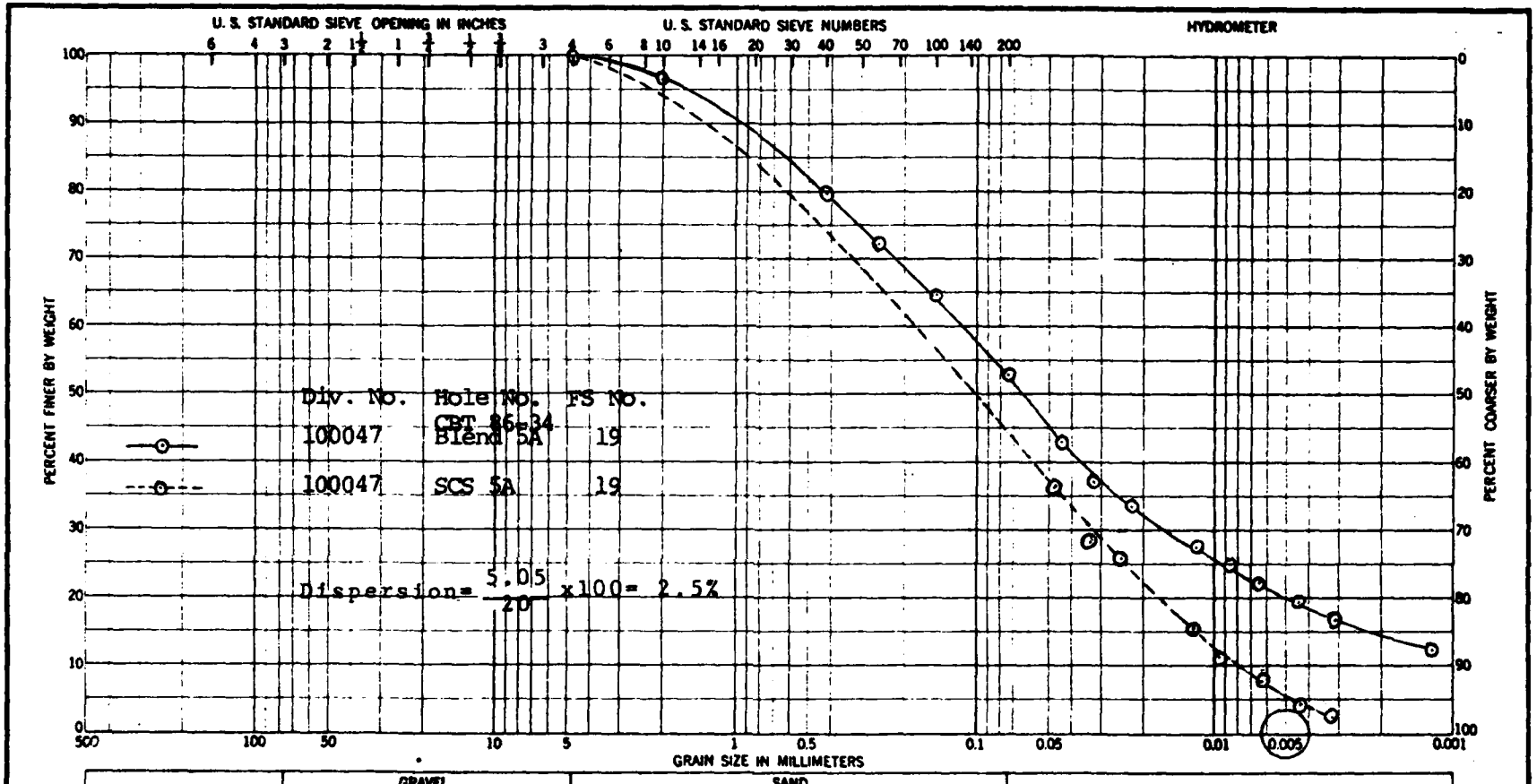
Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
17 & 18	0.75-5.0					

GRADATION CURVES

Project: UPPER SANTA ANA RIVER
SEVEN OAKS DAM
Area: Impervious Core (Phase II)
Boring No.
Date: August 1987

PLATE 18, PHASE II

ENG FORM MAY 63 2087



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
19	27.5-32.0					

GRADATION CURVES

Project: UPPER SANTA ANA RIVER
SEVEN OAKS DAM

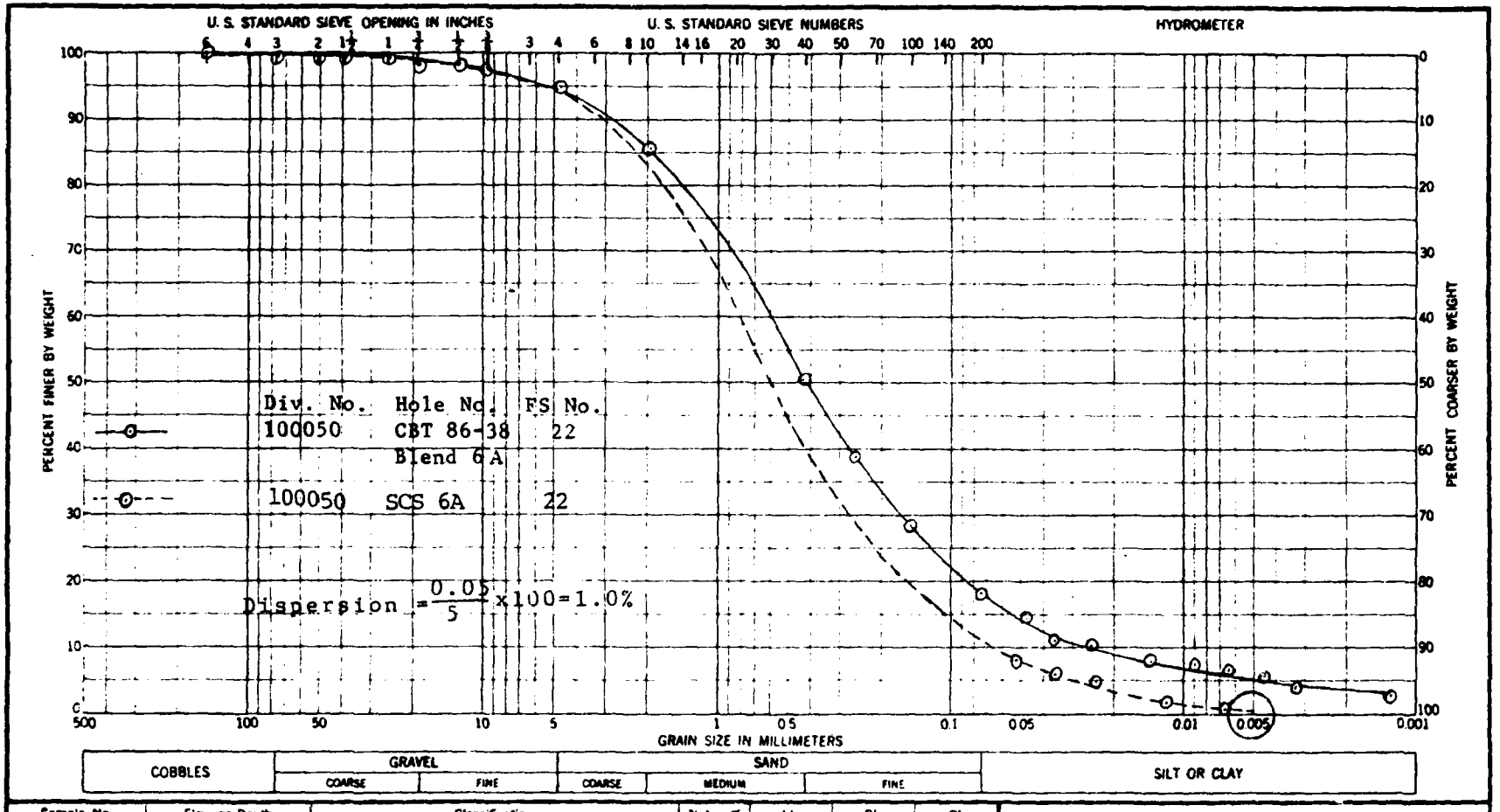
Area: Impervious Core (Phase II)

Boring No.

Date: August 1987

PLATE 15, PHASE II

ENG FORM 1 MAY 63 2087



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
22	7.0-12.0					

GRADATION CURVES

Project: Upper Santa Ana River
Seven Oaks Dam
Area: Impervious Core (Phase II)
Boring No:
Date: August 1987

PLATE 20, PHASE II

ENG FORM 1 MAY 63 2087

SPECIFIC GRAVITY

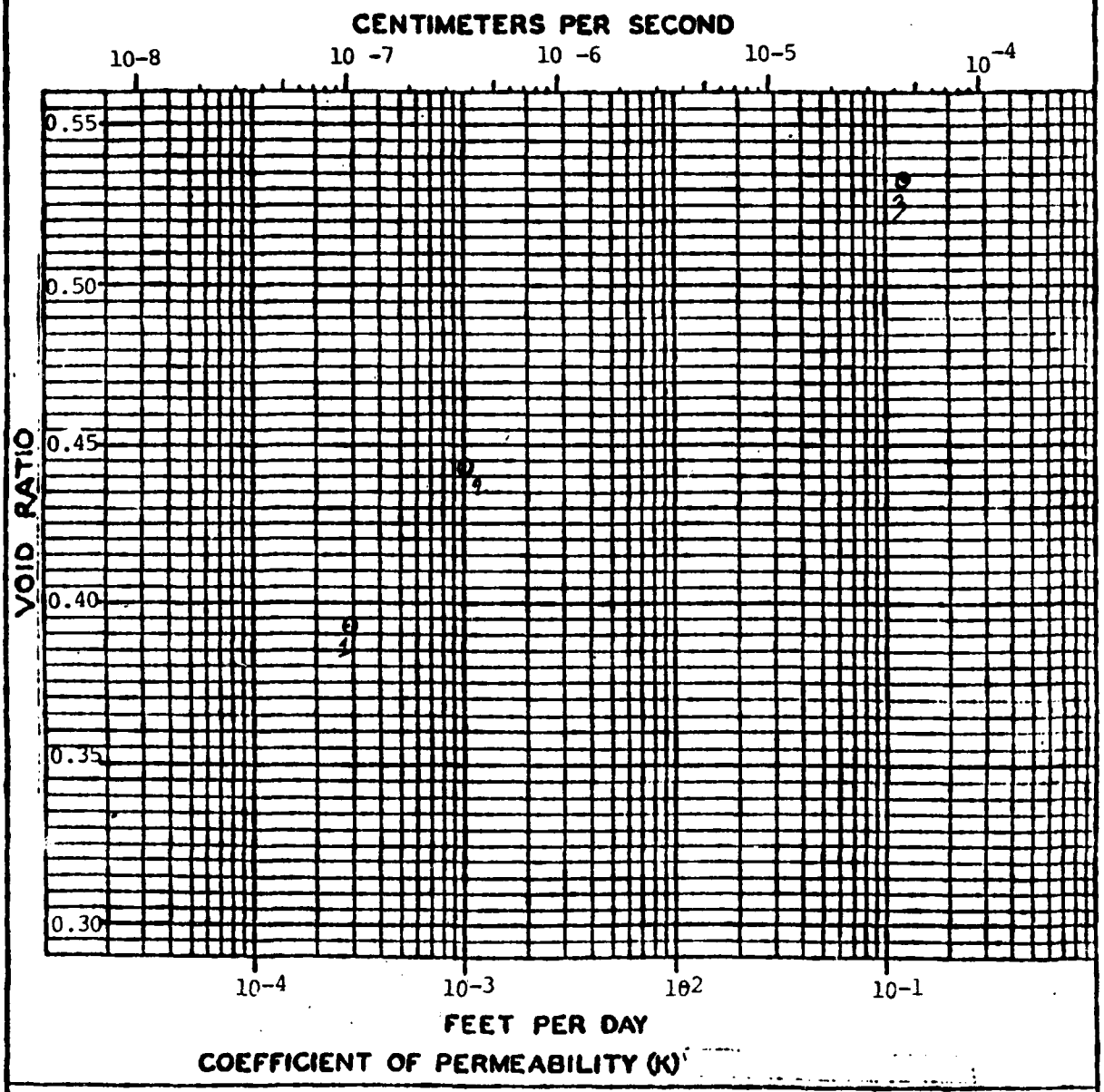
UPPER SANTA ANA - SEVEN OAKS DAM
(PHASE II)
March1987

Div. No.	Hole No.	F.S.	Elevation (ft.)	Specific Gravities +4	Specific Gravities -4
100029	CBT 86-30	1	0.75-5.0		2.68
100030	"	2	5.0-11.0	2.67	2.68
100031	CBT 86-31	3	1.0-2.5		2.68
100032	"	4	11.0-17.0	2.70	2.68
100033	CBT 86-32	5	1.0-3.0		2.70
100034	CBT 86-32	6	3.0-5.0		2.69

CORPS OF ENGINEERS U. S. ARMY

SOUTH PACIFIC DIVISION LABORATORY

PERMEABILITY



REMARKS: e_1=90%; e_2=95%; and e_3=100% Max Density, optimum moisture

Point	SPECIMEN DIAM. IN.	HT. IN.	MAX. PARTICLE	CONDITION
1	6.04	5.94	No. 4	Undisturbed
2	5.92	5.84		
3	5.84	5.76		

DISTRICT: Los Angeles

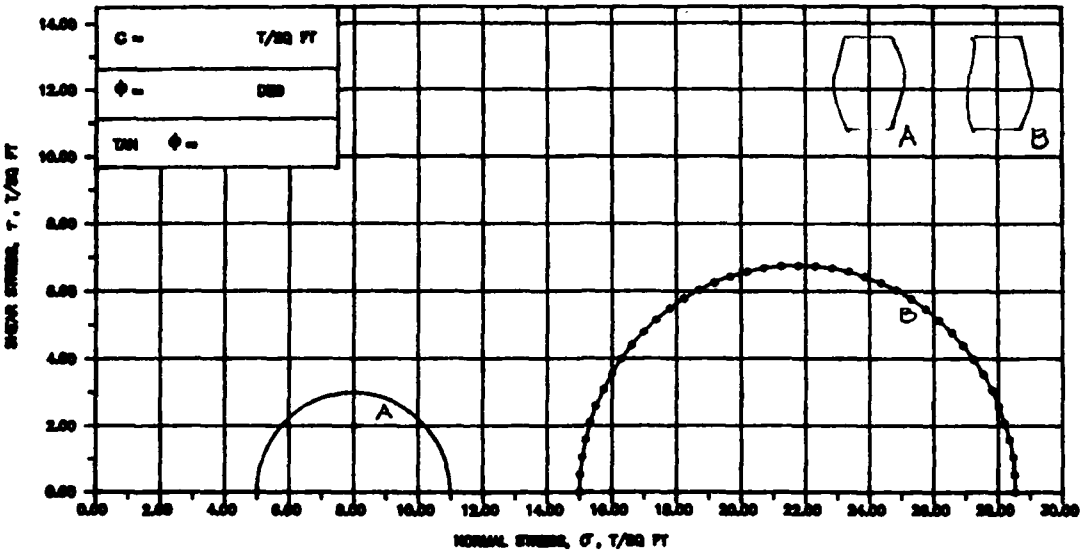
PROJECT: SEVEN OAKS DAM - Impervious Core (Phase II) (Blend 3A)

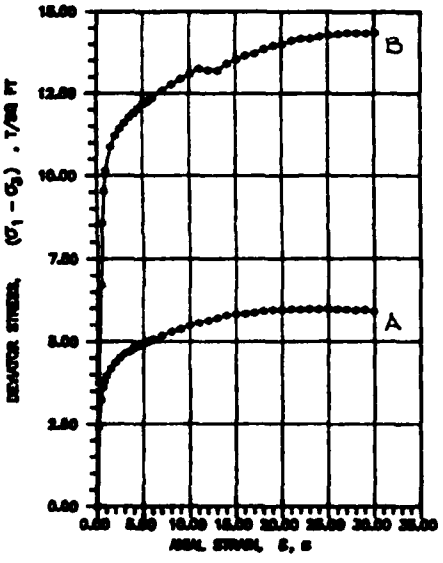
Point	DIV. NO.	HOLE NO.	F.S. NO.	DEPTH FROM	DEPTH TO
1	100045	CBT-86-34	17	0.75	5
2	100046	" " "	18	0.75	5
3					

TESTED	COMPUTED	DRAWN	CHECKED
FN	FN	CRK	DW

SPD FORM 6 OCT 55 67 (CIVIL) REPLACES PSD 67, (CIVIL), 20 JULY 49, WHICH MAY BE USED.

PLATE 22 PHASE II





SPECIMEN NO.			A	B
INITIAL	WATER CONTENT, %	w_o	13.0	13.0
	DRY DENSITY LB/CU FT	γ_{d_o}	121.0	121.1
	SATURATION, %	S_o	90	90
	VOID RATIO	e_o	0.393	0.392
BEFORE SHEAR	WATER CONTENT, %	w_c	12.9	12.0
	DRY DENSITY LB/CU FT	γ_{d_c}	125.0	127.3
	SATURATION, %	S_c	100	100
	VOID RATIO	e_c	0.348	0.324
	PORE BACK PRESSURE, T/SQ FT	u_b	6.5	5.8
CELL PRESSURE, T/SQ FT		σ_3	5.0	15.0
DEVIATOR STRESS, T/SQ FT	$(\sigma_1 - \sigma_3)_{MAX}$		5.82	13.54
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	322	104
ULTIMATE DEVIATOR STRESS, T/SQ FT	$(\sigma_1 - \sigma_3)_{ULT}$		-	-
INITIAL DIAMETER, IN.		D_o	2.80	2.80
INITIAL HEIGHT, IN.		H_o	6.45	6.45

CONTROLLED STRAIN TEST: Yes

DESCRIPTION OF SPECIMEN: Sandy Clay (CL)

LL 27	PL 16	PI 11	G_s 2.70

TYPE OF SPECIMEN: Remolded | TYPE OF TEST: "R"

REMARKS: Div.No. 100045&100046
Remolded to 98% max.density
@2% above Opt.water
max.density = 120.8 pcf
Opt.water = 12.2%

PROJECT: Seven Oaks Dam
Impervious Borrow

BORING NO. CBT 86-34 | SAMPLE NO. 17&18

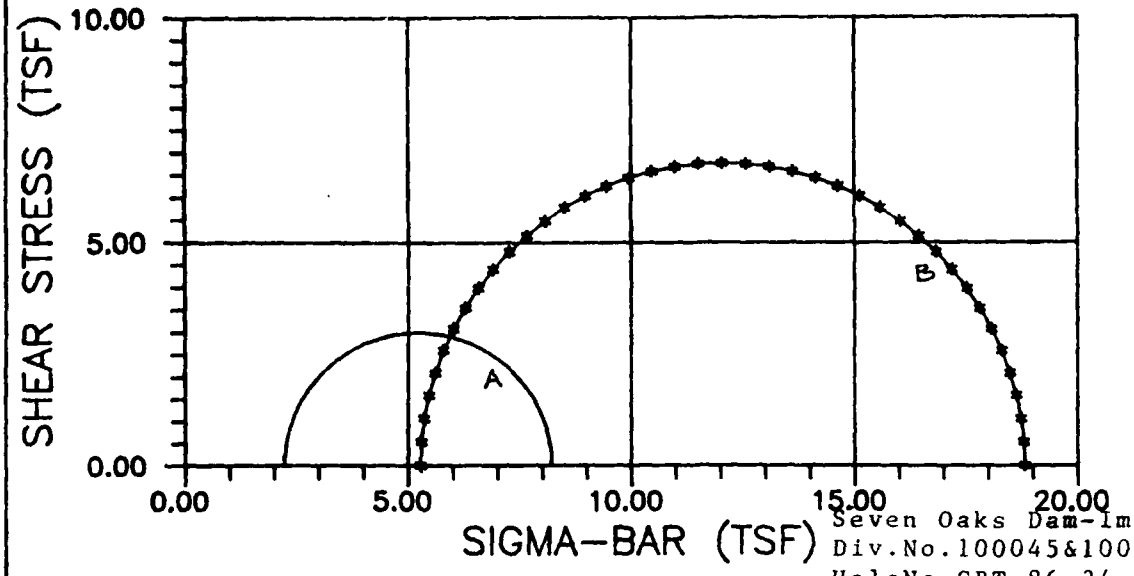
Blend 3A

LABORATORY: SPD | DATE: November 1987

TRIAXIAL COMPRESSION TEST REPORT

DOD FORM NO. 2089
REV. JUNE 1970

PLATE 23, PHASE II



Seven Oaks Dam-Impervious
Div.No.100045&100046
HoleNo.CBT 86-34
F.S.No.17&18,Blend 3A

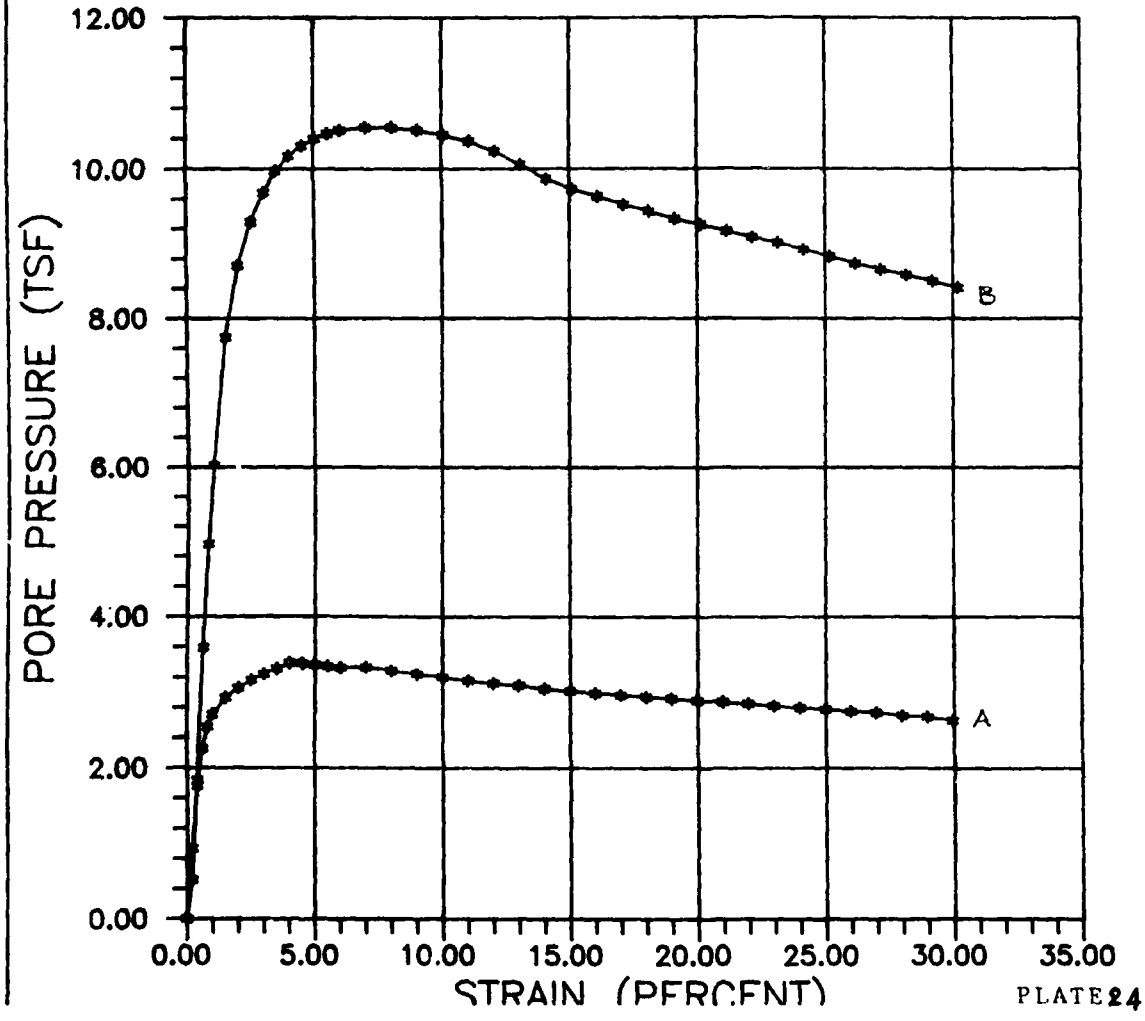
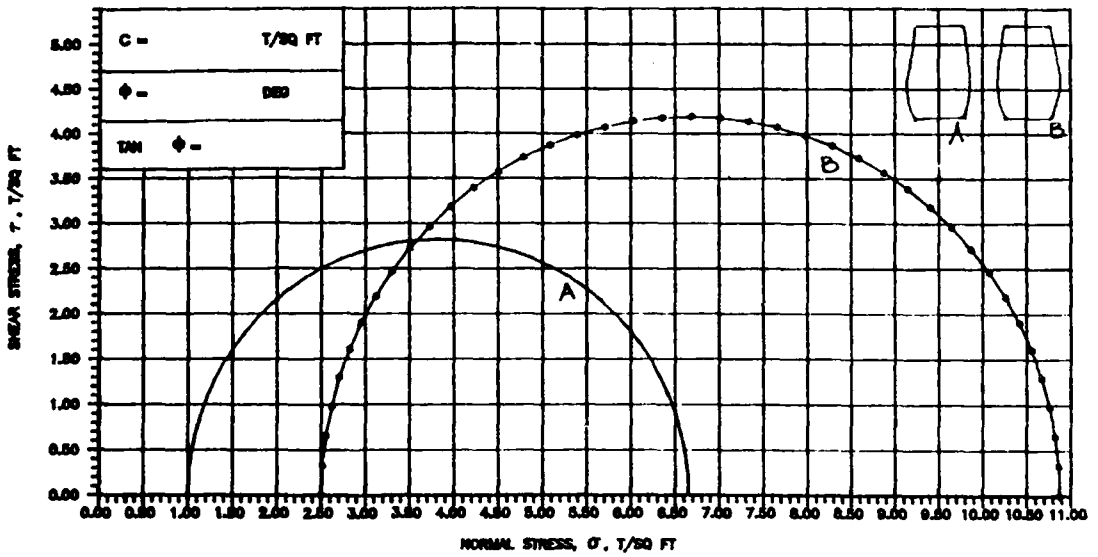
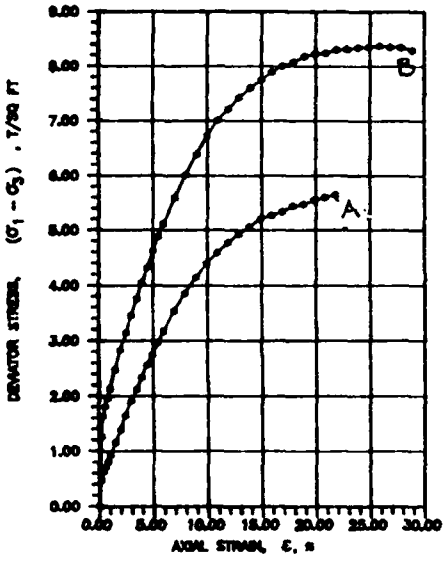


PLATE 24, PHASE II





SPECIMEN NO.			A	B
INITIAL	WATER CONTENT, %	w_o	13.2	13.3
	DRY DENSITY LB/CU FT	γ_{d_o}	120.7	120.7
	SATURATION, %	S_o	90	91
	VOID RATIO	e_o	0.396	0.396
BEFORE SHEAR	WATER CONTENT, %	w_c	13.4	12.7
	DRY DENSITY LB/CU FT	γ_{d_c}	123.8	125.5
	SATURATION, %	S_c	100	100
	VOID RATIO	e_c	0.361	0.343
	FINAL BACK PRESSURE, T/SQ FT	u_o	5.8	5.8
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	1.0	2.5
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	5.24	7.75
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	243	247
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$	-	-
INITIAL DIAMETER, IN.		D_o	2.80	2.80
INITIAL HEIGHT, IN.		H_o	6.55	6.55

CONTROLLED STRAIN TEST: Yes

DESCRIPTION OF SPECIMEN: Sandy Clay (CL)

LL 27	PL 16	PI 11	G_s 2.70

TYPE OF SPECIMEN: Remolded | TYPE OF TEST: "R"

REMARKS: Div.No. 100045 & 100046
Remolded to 98% max.density
@ 2% above opt.water
max.density = 120.8 pcf
Opt.water = 12.2%

PROJECT: Seven Oaks Dam
Impervious Borrow

BORING NO. CBT 86-34 | SAMPLE NO. 17& 18

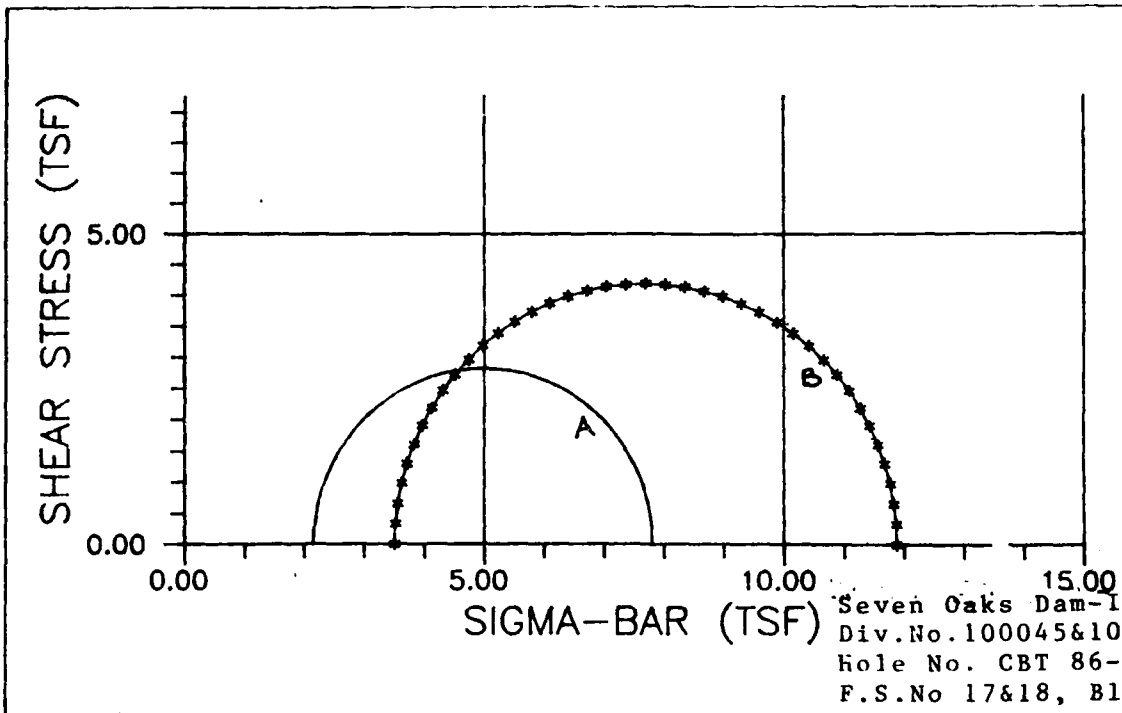
Blend 3A

LABORATORY SPD | DATE November1987

TRIAXIAL COMPRESSION TEST REPORT

USBR FORM NO. 3909
REV. JUNE 1970

PLATE 25, PHASEII



Seven Oaks Dam-Impervious
Div.No.100045&100046
Hole No. CBT 86-34
F.S.No 17&18, Blend 3A

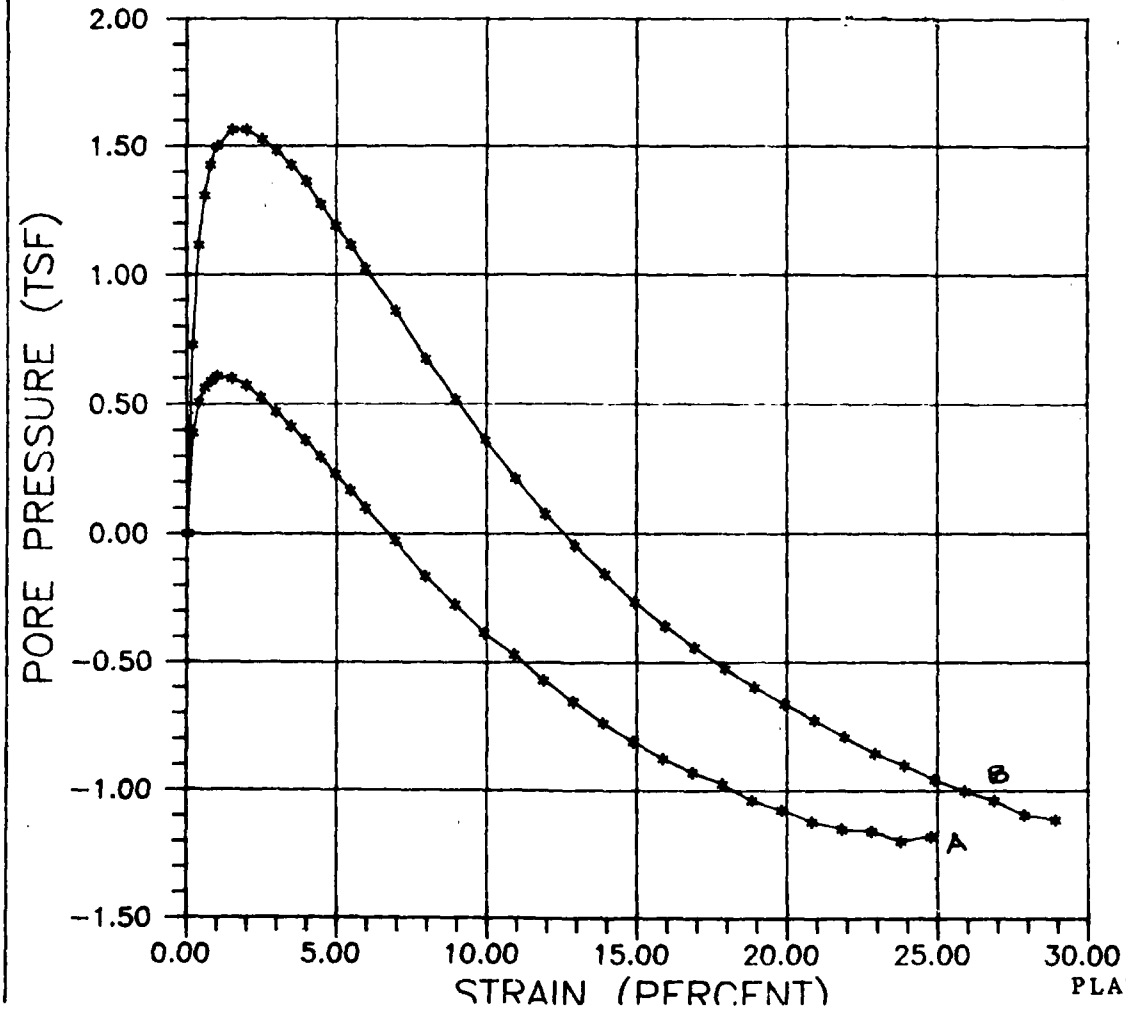
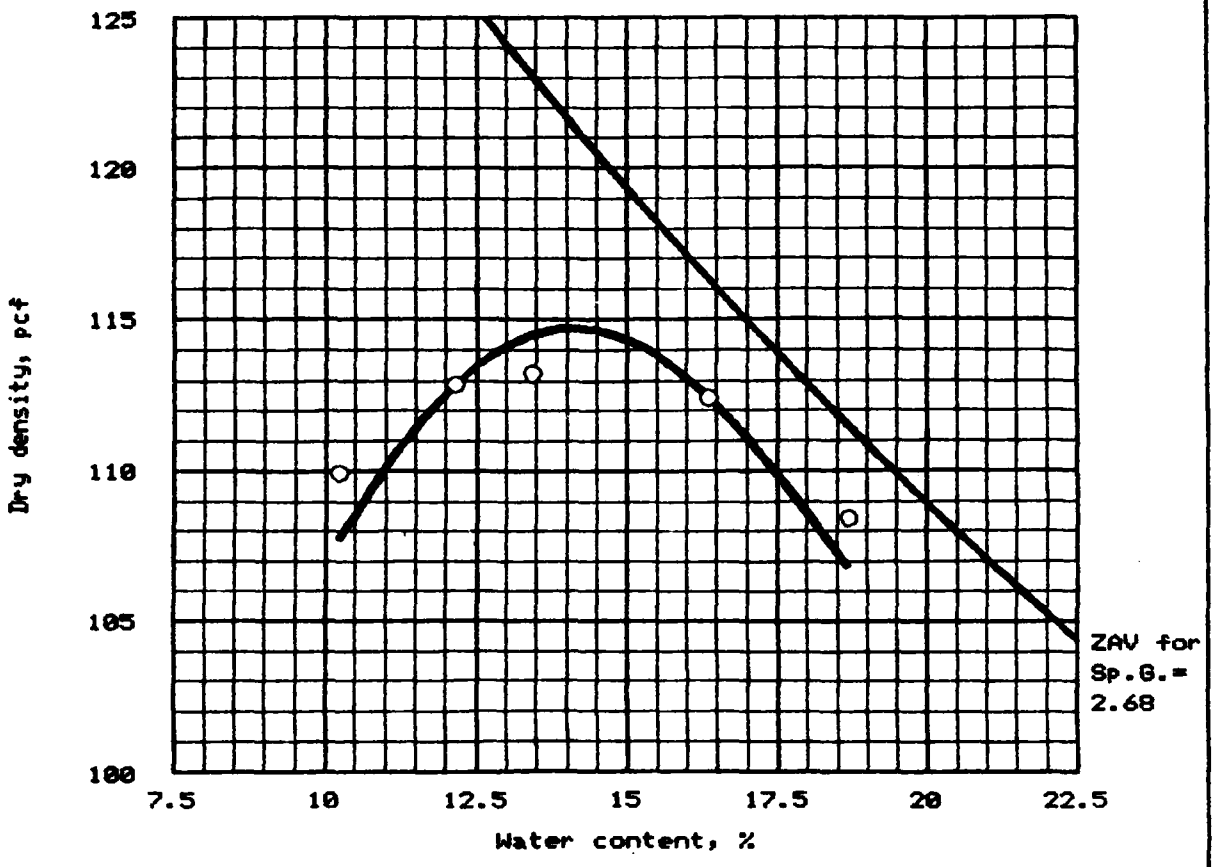


PLATE 26, PHASE II

COMPACTION TEST REPORT



"Standard" Proctor, ASTM D 698-78, Method A

Elev/ Depth	USCS Classification	Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
0.75-5	Silty Clayey Sand (SC)	3.3 %	2.68	33	11	0 %	34 %

TEST RESULTS	MATERIAL DESCRIPTION
Optimum moisture = 14.1 % Maximum dry density = 114.7 pcf	Remolded

Division No.: 100029 F.S. No. 1
Project: 7 Oaks Dam
Location: Impervious Core
Hole No. CBT-86-30
Date: 8-25-1987

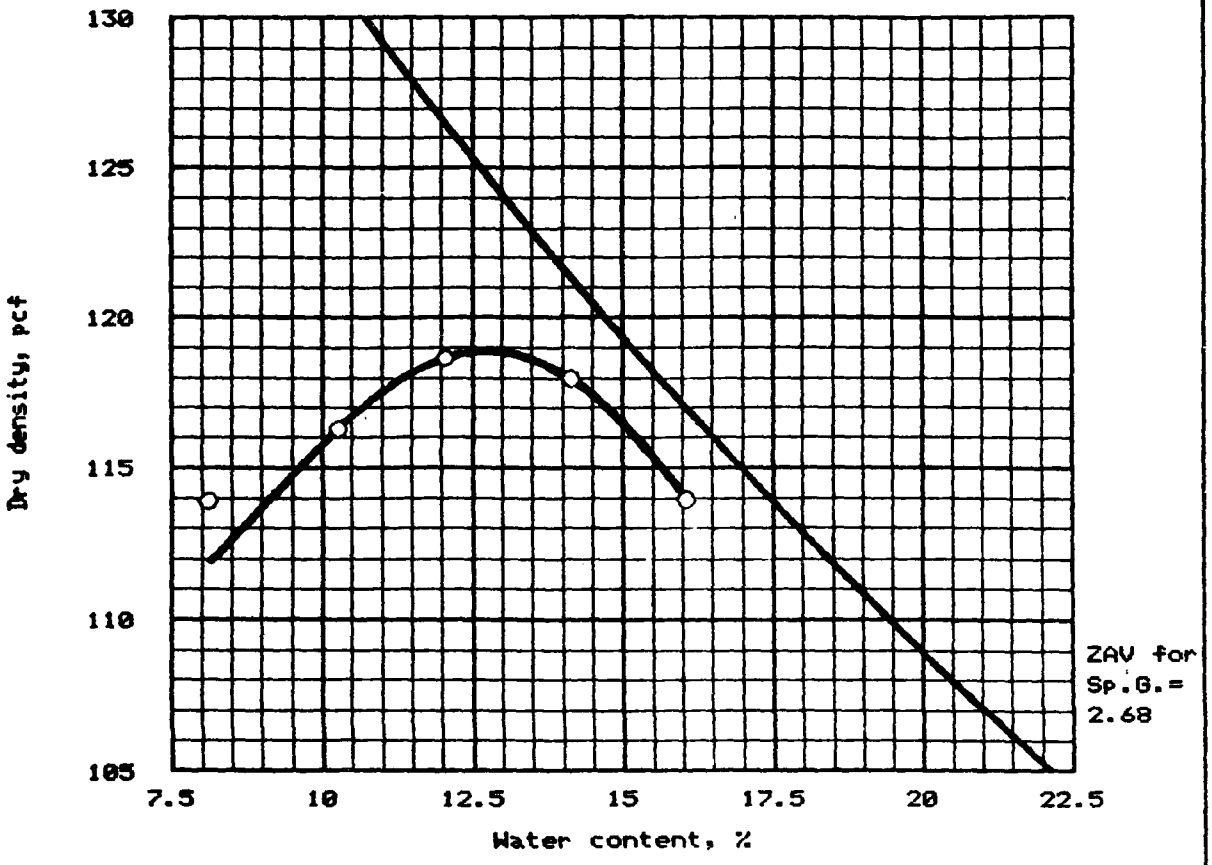
Remarks:
1A
Barrel No. 1
(Phase II)

COMPACTION TEST REPORT
CORPS OF ENGINEERS - SPDL

Plate No. 27

PHASE II

COMPACTION TEST REPORT



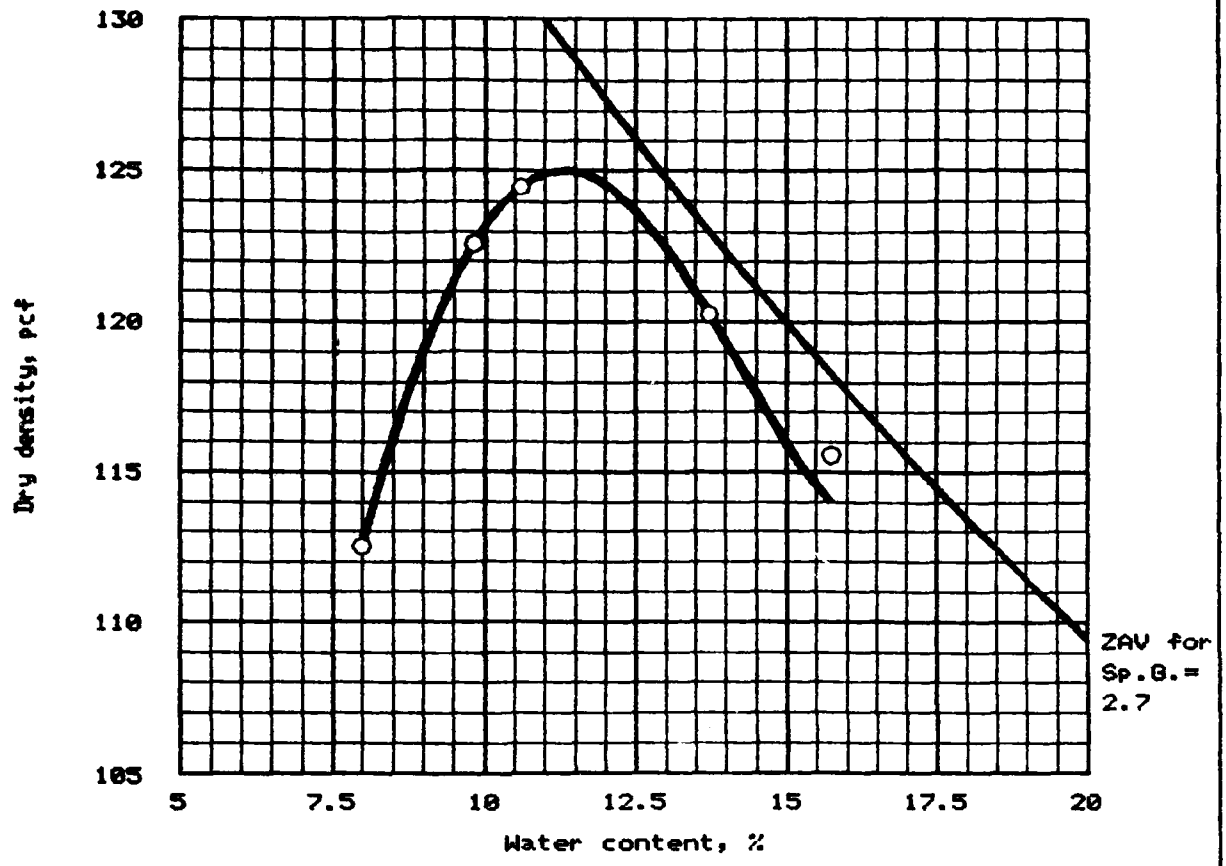
"Standard" Proctor, ASTM D 698-78, Method A

Elev/ Depth	USCS Classification	Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
5-11	Clayey Sand(SC)	0.9 %	2.68	33	13	3 %	24 %

TEST RESULTS	MATERIAL DESCRIPTION
Optimum moisture = 12.7 % Maximum dry density = 118.9 pcf	Remolded
Division No.: 100030 F.S. No. 2 Project: 7 Oaks Dam Location: Impervious Core Hole No. CBT-86-30 Date: 8-21-1987	Remarks: 2A Barrel No. 2 (Phase II)
COMPACTION TEST REPORT CORPS OF ENGINEERS - SPDL	Plate No. 2f

PHASE II

COMPACTION TEST REPORT



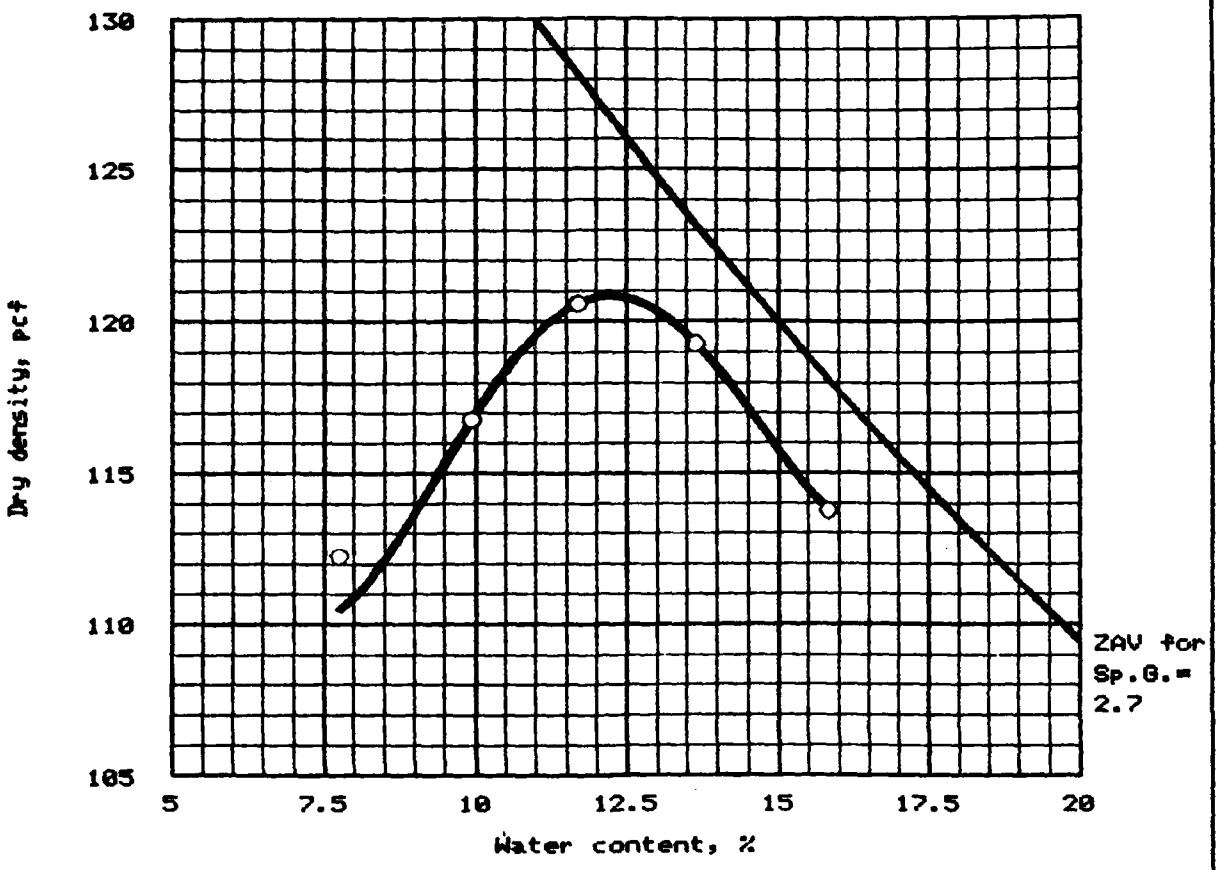
"Standard" Proctor, ASTM D 698-78, Method A

Elev/ Depth	USCS Classification	Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
0.75-5	Clayey Sand (SC)	9.8 10.9	2.7	27	11	0 %	52 %

TEST RESULTS	MATERIAL DESCRIPTION
Optimum moisture = 11.3 % Maximum dry density = 125.0 pcf	Remolded
Division No.: 100045 & 100046 F.S. No. 17/18 Project: 7 Oaks Dam Location: Impervious Core Hole No. CBT-86-34 Date: 8-21-1987	Remarks: Blend 3A Barrel No. 17/18 (Phase II)
COMPACTION TEST REPORT CORPS OF ENGINEERS - SPDL	Plate No. 29

PHASE II

COMPACTION TEST REPORT



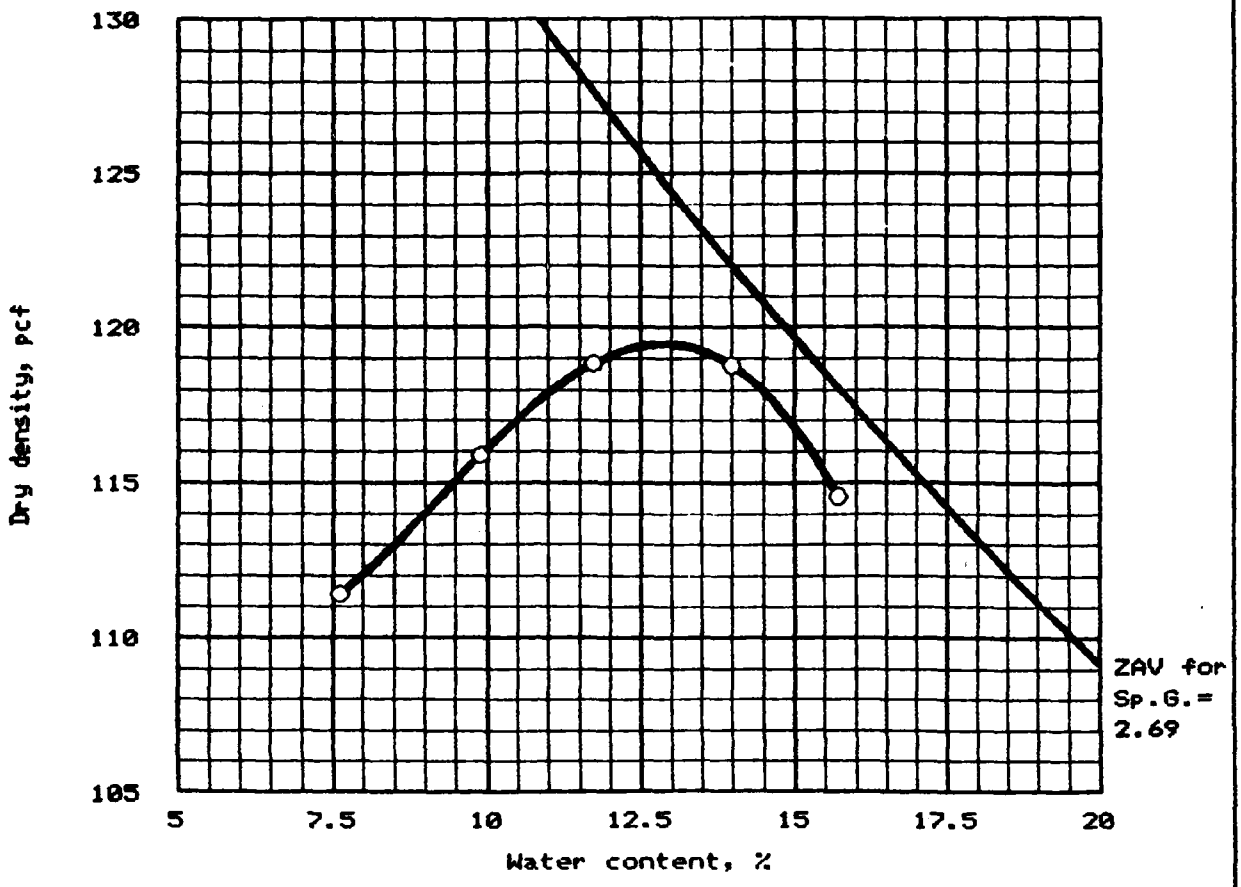
"Standard" Proctor, ASTM D 698-78, Method A

Elev/ Depth	USCS Classification	Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
.75-5	Claeyey Sand(SC) Sandy Clay (CL)	9.8 10.9	2.70	27	11	0 %	52 %

TEST RESULTS	MATERIAL DESCRIPTION
Optimum moisture = 12.2 % Maximum dry density = 120.8 pcf	REMOLDED
Division No.: 100045 & 100046 F.S. No. 17 & 18 Project: 7 Oaks Dam Location: Impervious Core Hole No. CBT-86-34 Date: 8-21-1987	Remarks: Blend 3A Barrel No. 17/18 (Phase II)
COMPACTION TEST REPORT CORPS OF ENGINEERS - SPDL	Plate No. 30

PHASE II

COMPACTION TEST REPORT



"Standard" Proctor, ASTM D 698-78, Method A

Elev/ Depth	USCS Classification	Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
27-32	Sandy Clay(CL)	12.1 %	2.69	27	11	0 %	53 %

TEST RESULTS	MATERIAL DESCRIPTION
Optimum moisture = 12.9 % Maximum dry density = 119.5 pcf	Remolded

Division No.: 100047 FS No. 19
Project: 7 Oaks Dam
Location: Impervious Core
Hole No. CBT-86-34
Date: 8-21-1987

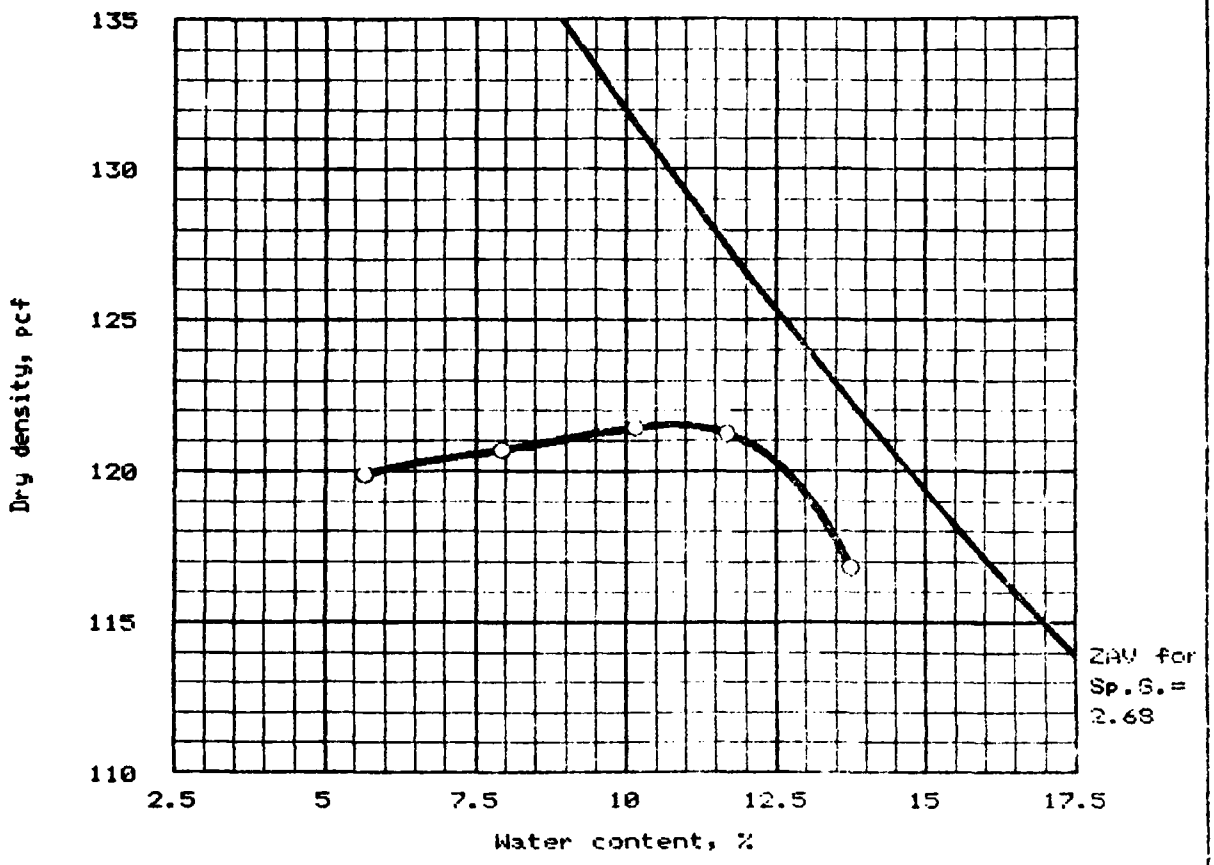
Remarks:
5A
Barrel No. 19
(Phase II)

COMPACTION TEST REPORT
CORPS OF ENGINEERS - SPDL

Plate No. 31

PHASE II

COMPACTION TEST REPORT



"Standard" Proctor, ASTM D 698-78, Method A

Elev./ Depth	USCS Classification	Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
7-12	Silty Sand (SM)	5.6 %	2.68			5 %	18 %

TEST RESULTS	MATERIAL DESCRIPTION
Optimum moisture = 10.7 % Maximum dry density = 121.5 pcf	Remolded

Division No.: 100050 F.S. No. 22
Project: 7 Oaks Dam
Location: Impervious Core
Hole No. CBT-86-38
Date: 9-02-1987

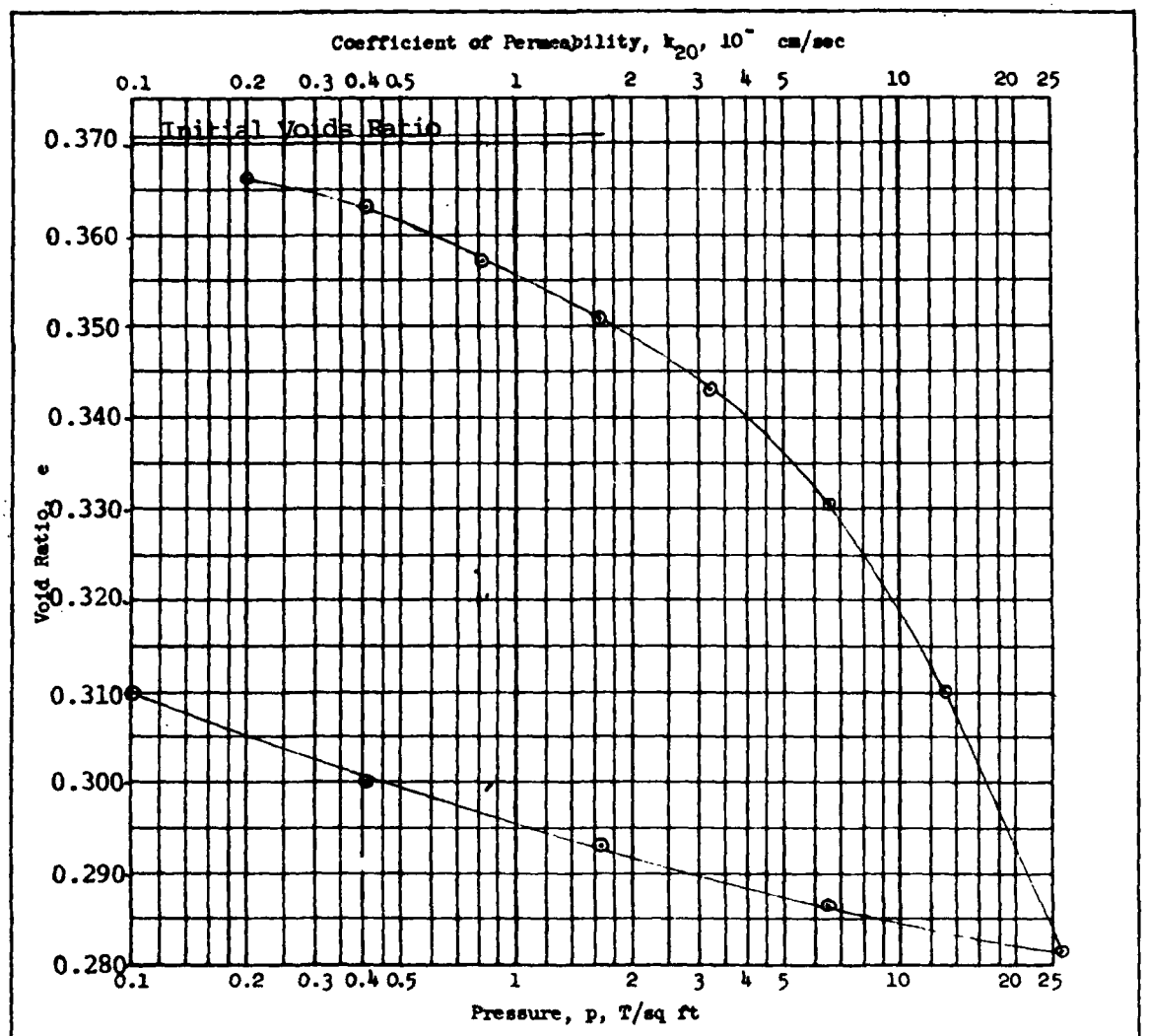
Remarks:
6A
Barrel No. 22
(Phase II)

COMPACTION TEST REPORT

CORPS OF ENGINEERS - SOUTHWESTERN

Plate No. 32

PHASE II

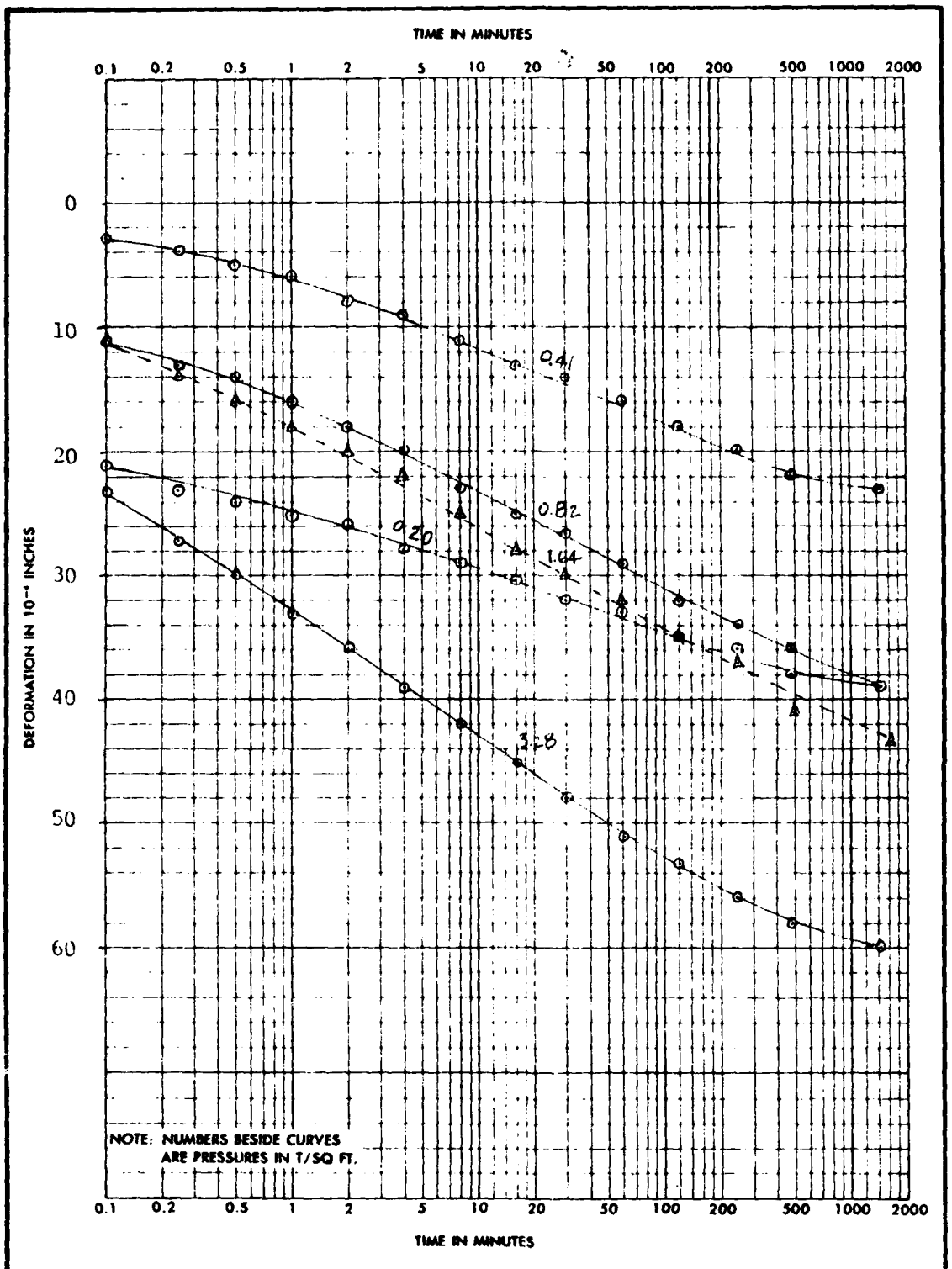


Type of Specimen	Remolded			Before Test		After Test	
Diam 4.4 in.	Ht 1.0 in.			Water Content, w_o	12.9 %	w_f	11.6 %
Overburden Pressure, p_o	T/sq ft			Void Ratio, e_o	0.371	e_f	0.313
Preconsol. Pressure, p_c	T/sq ft			Saturation, S_o	94 %	S_f	100 %
Compression Index, C_c				Dry Density, γ_d	122.9 lb/ft^3		
Classification	Sandy Clay (CL)			k_{20} at e_o =	x 10^{-} cm/sec		
LL 27	G_s 2.70			Project	SEVEN OAKS DAM		
PL 16	D_{10}				Impervious Core		
Remarks	Remolded to 101% max.			Area	FS No. 17 & 18 (PHASE II)		
density @ Opt water content.				Boring No.	CBT-86-34	Sample No.	Blend 3A
Div.No. 100045&100046				Depth El	0.75' - 5'	Date	July 1987

CONSOLIDATION TEST REPORT

ENG FORM 2090 (MAY 63) PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT)

PLATE 33 of 3424 PHASE II



PROJECT SEVEN OAKS DAM - IMPERVIOUS CORE

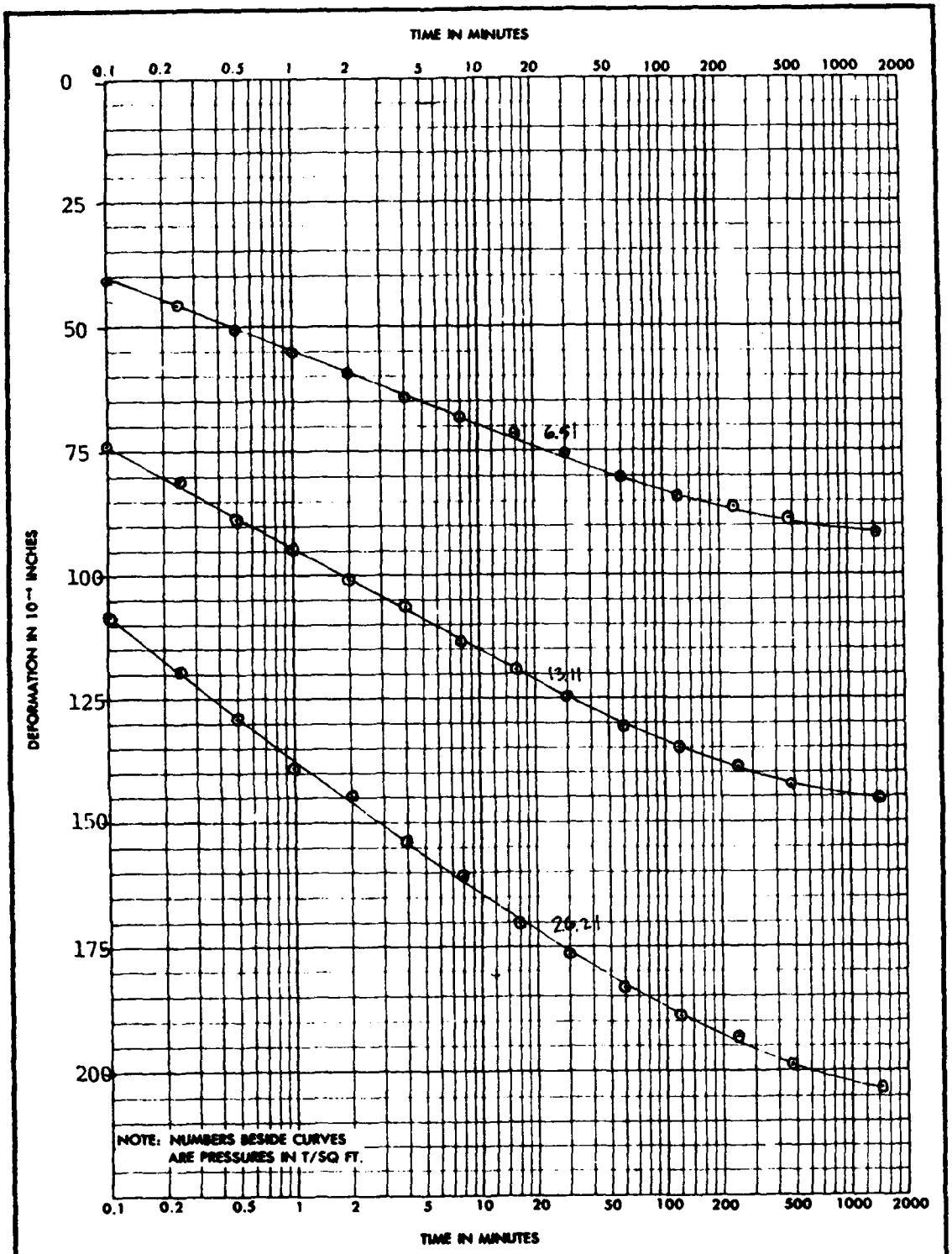
AREA Div.No.100045&46 FS No. 17 & 18 (PHASE II)

BORING NO.	SAMPLE NO.	DEPTH EL	DATE
CBT-86-34	Blend 3A	0.75'-5.0'	July 1987

ENG FORM 2088 1 MAY 63 PREVIOUS EDITIONS ARE OBSOLETE.

CONSOLIDATION TEST—TIME CURVES (TRANSLUCENT)

PLATE 34, PHASE II

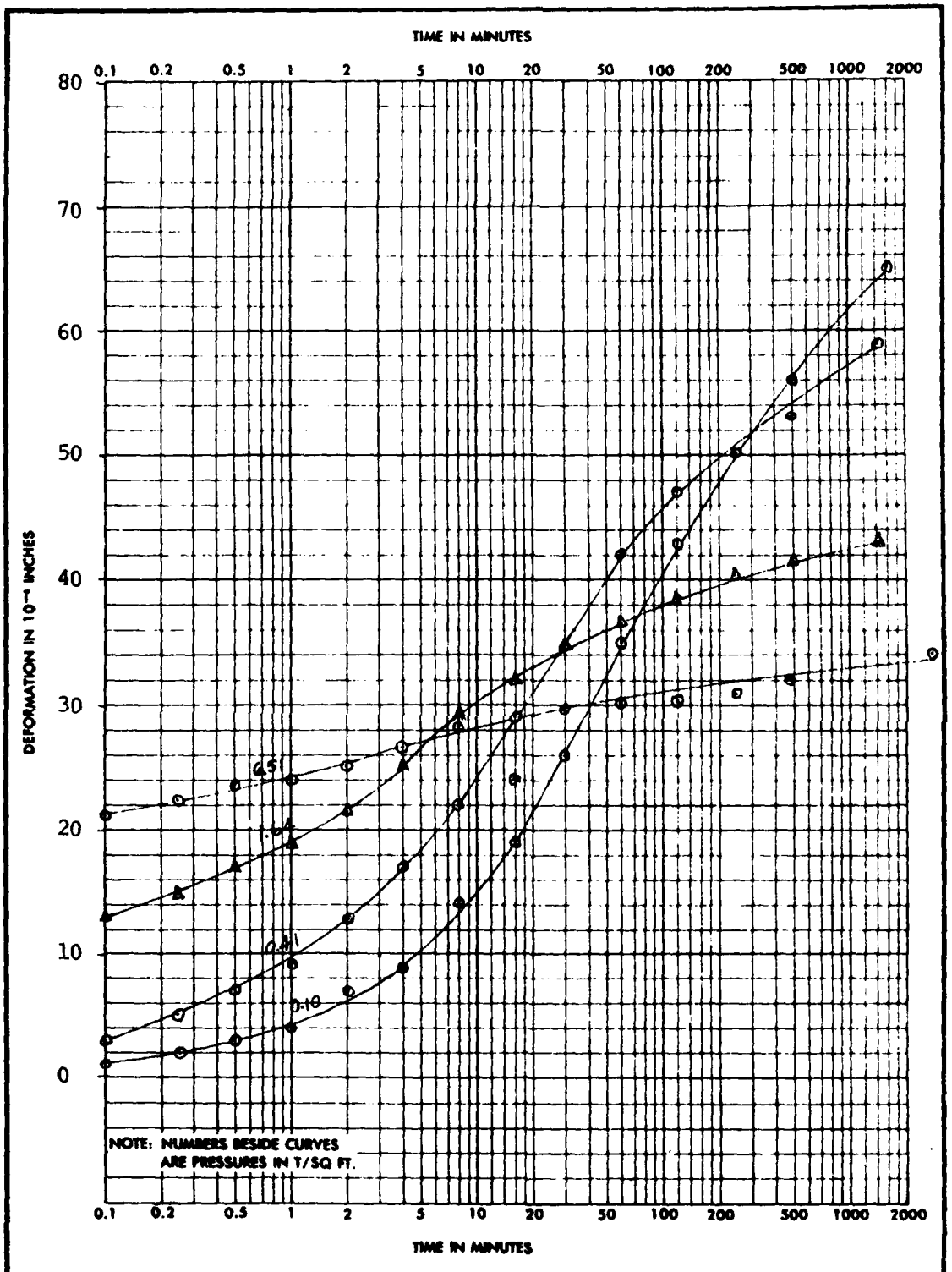


PROJECT	SEVEN OAKS DAM - IMPERVIOUS CORE			
AREA	Div.No.100045&100046 FS No. 17 & 18 (PHASE II)			
BORING NO.	CBT-86-34	SAMPLE NO. Blend 3A	DEPTH EL 0.75'-5.0'	DATE July 1987

ENG FORM 2088 1 MAY 63 PREVIOUS EDITIONS ARE OBSOLETE.

CONSOLIDATION TEST—TIME CURVES (TRANSLUCENT)

PLATE 35, PHASE II

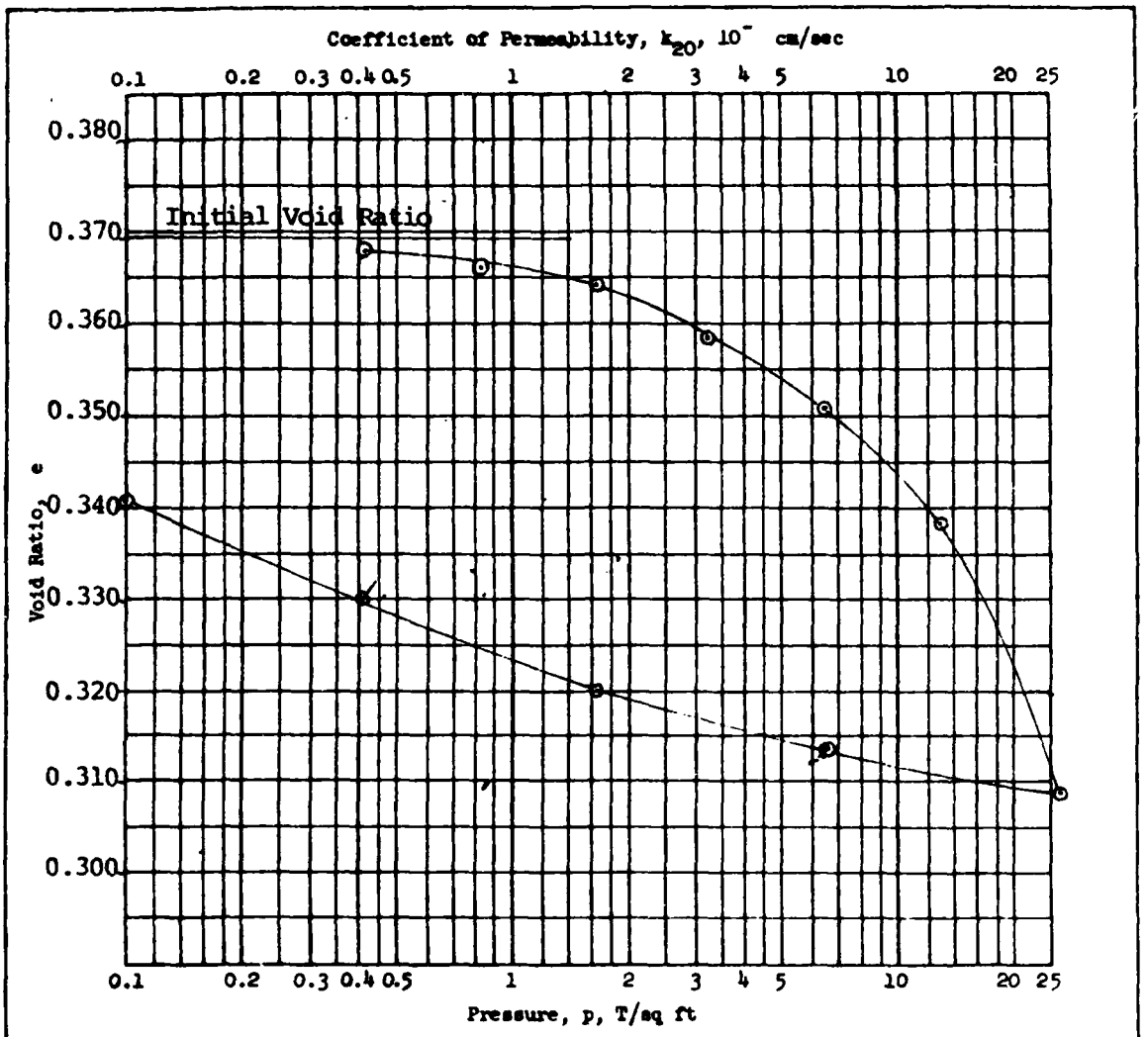


PROJECT	SEVEN OAKS DAM - IMPERVIOUS CORE (PHASE II)				
AREA	FS No. 17 & 18 Div.No. 100045 & 100046				
BORING NO. CBT-86-34	SAMPLE NO. Blend 3A	DEPTH EL	0.75'-5.0'	DATE	July 1987

ENG FORM 2088 1 MAY 63 PREVIOUS EDITIONS ARE OBSOLETE.

CONSOLIDATION TEST—TIME CURVES (TRANSLUCENT)

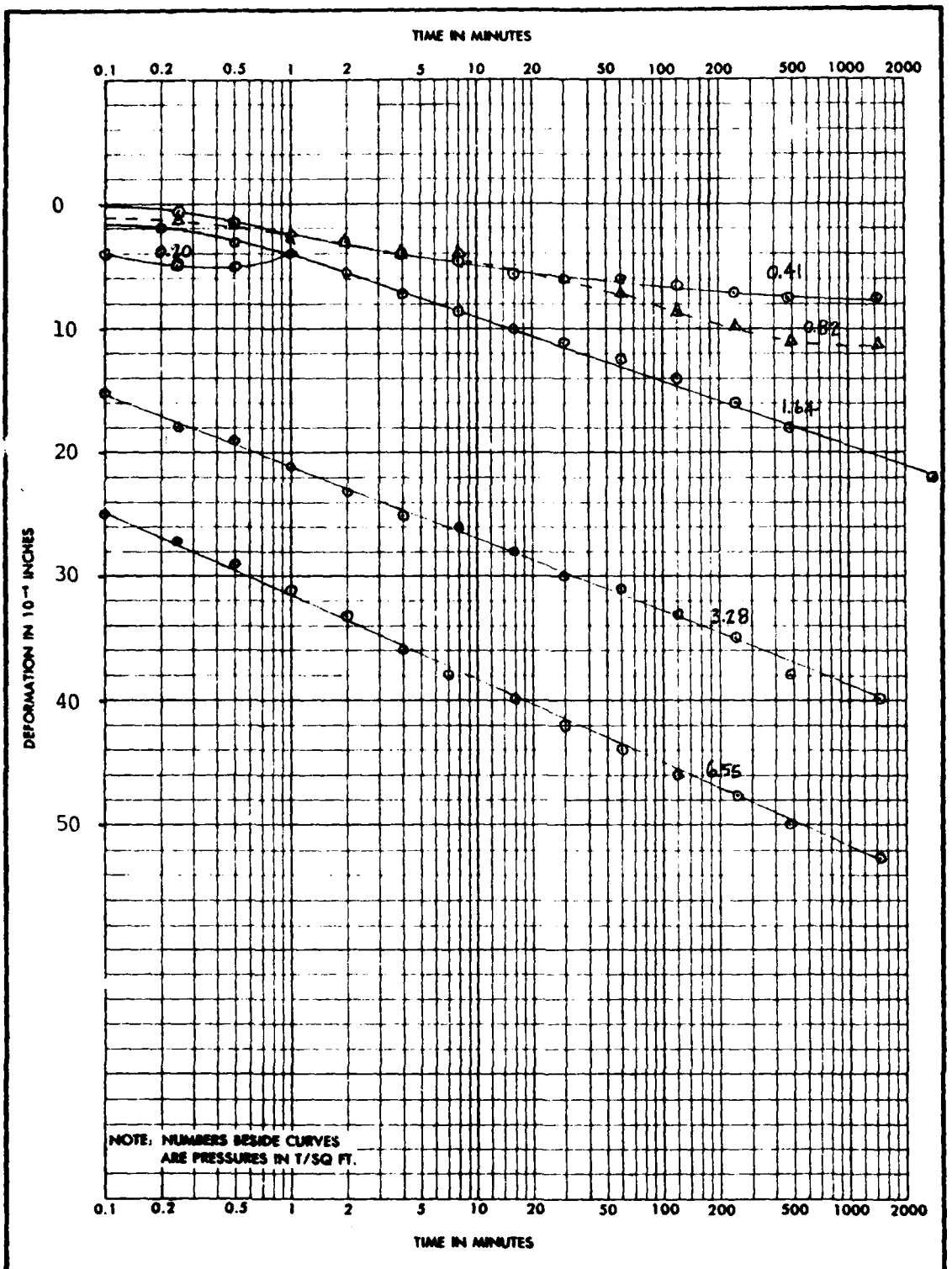
PLATE 36, PHASE II



Type of Specimen: Remolded		Before Test		After Test	
Diam 4.4 in.	Ht 10 in.	Water Content, w_o	10.7 %	w_f	13.4 %
Overburden Pressure, Po T/sq ft		Void Ratio, e_o	0.369	e_f	0.365
Preconsol. Pressure, Pc T/sq ft		Saturation, S_o	78 %	S_f	99 %
Compression Index, C_c		Dry Density, γ_d	123.0 lb/ft^3		
Classification: Sandy Clay (CL)		k_{20} at e_o =	× 10^{-} cm/sec		
LL 27	G_s 2.70	Project: SEVEN OAKS DAM			
PL 16	D_{10}	Impervious Core			
Remarks: Remolded to 101% max. density @ opt - 2% water content.		Area: FS No. 17 & 18 (Phase II)			
		Boring No. CBT-86-34	Sample No. Blend 3A		
		Depth El 0.75' - 5'	Date July 1987		
Div.No.100045&100046		CONSOLIDATION TEST REPORT			

ENG FORM 1 MAY 63 2090 PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT)

PLATE 37, PHASE II

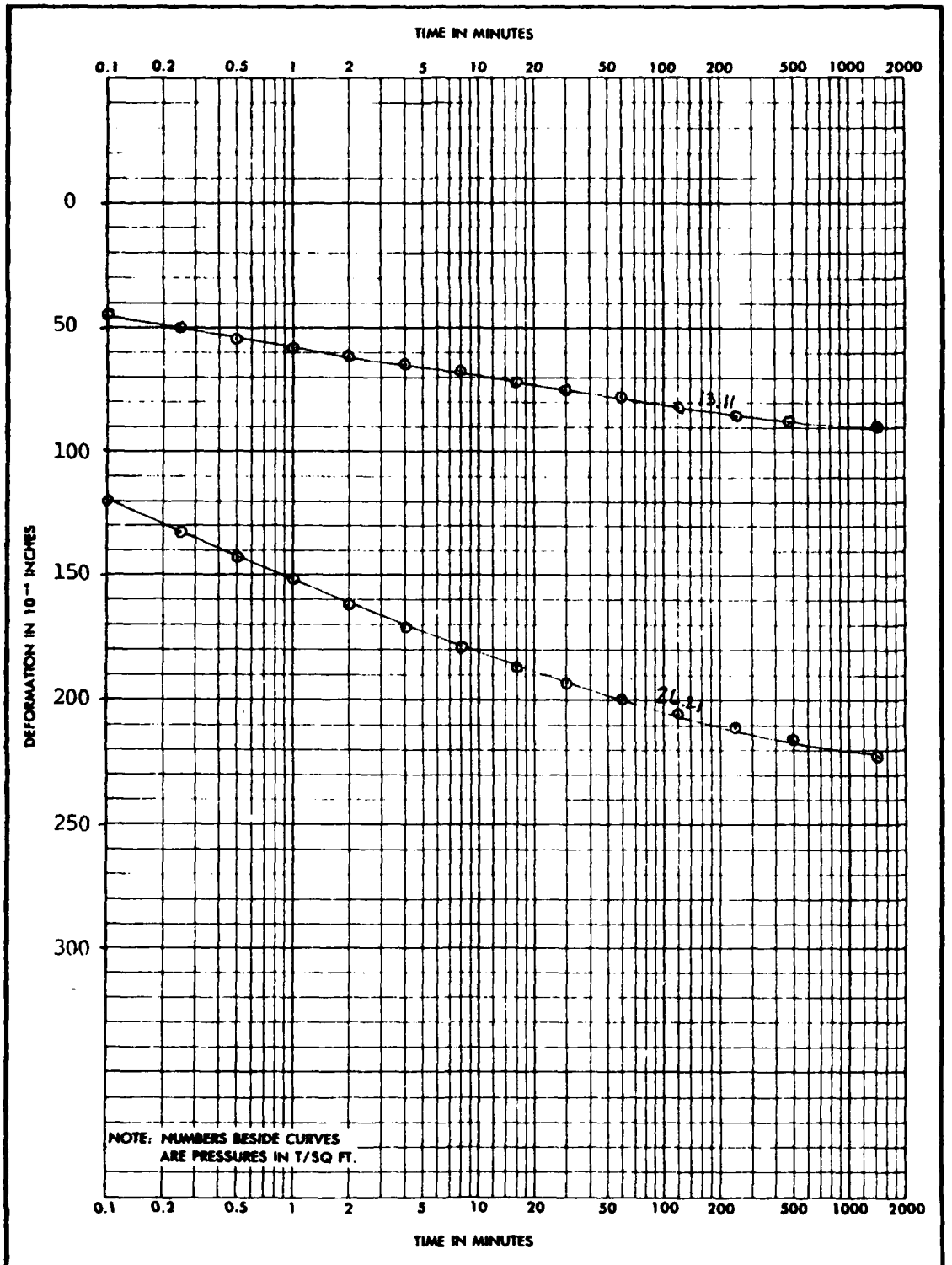


PROJECT	SEVEN OAKS DAM - IMPERVIOUS CORE			
AREA	Div.No.100045& 46	fS No. 17 & 18	(Phase I)	
BORING NO. CBT-86-34	SAMPLE NO. Blend 34	DEPTH EL 0.75'-5.0'	DATE July 1987	

ENG FORM 2088 1 MAY 63 PREVIOUS EDITIONS ARE OBSOLETE.

CONSOLIDATION TEST—TIME CURVES (TRANSLUCENT)

PLATE 38, PHASE II

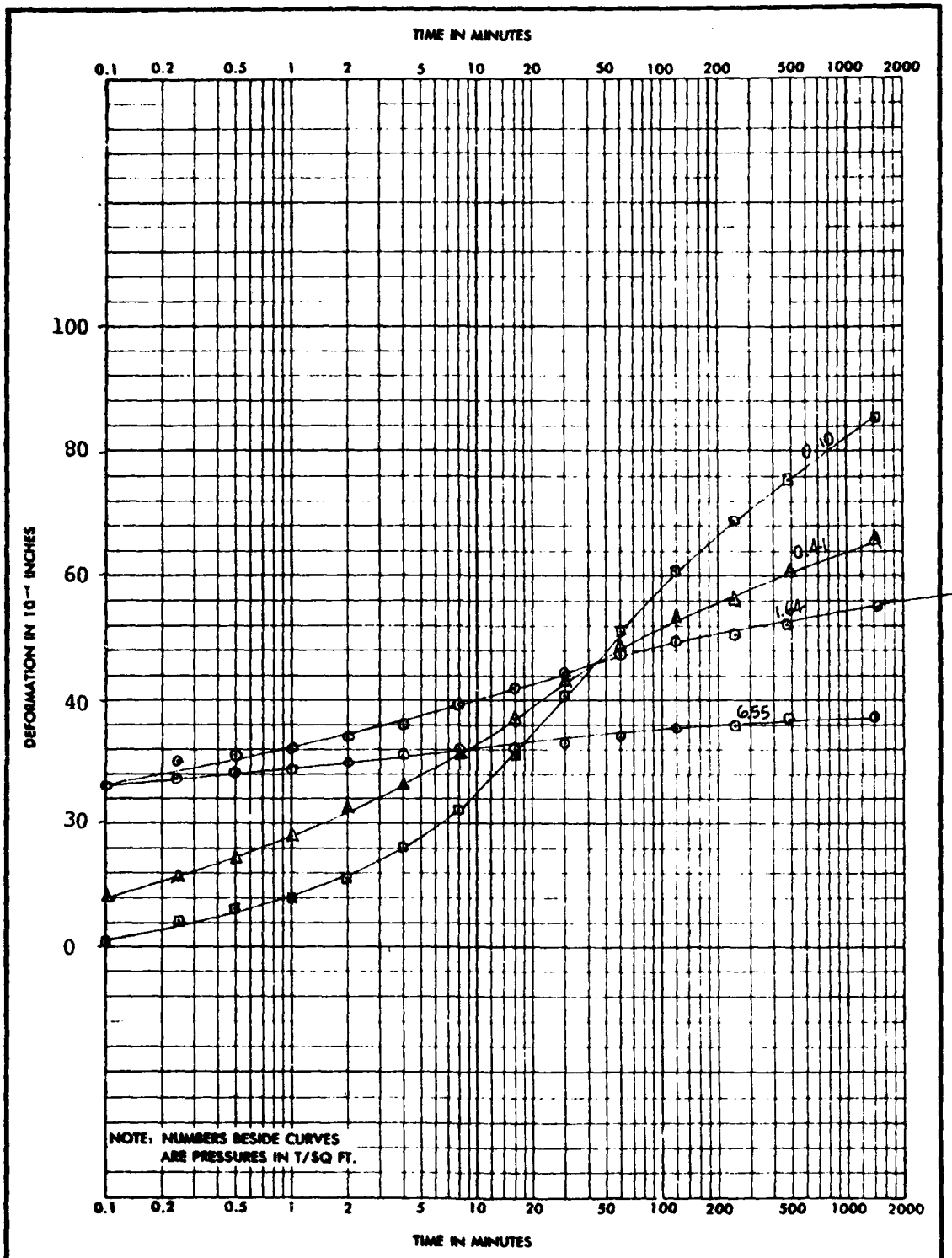


PROJECT	SEVEN OAKS DAM - IMPERVIOUS CORE				
AREA	Div.No. 100045 & 46 FS No. 17 & 18 (Phase II)				
BORING NO.	CBT-86-34	SAMPLE NO.	Blend 34	DEPTH EL	0.75'-5.0'
DATE	July 1987				

ENG FORM 2088 1 MAY 63 PREVIOUS EDITIONS ARE OBSOLETE.

CONSOLIDATION TEST—TIME CURVES (TRANSLUCENT)

PLATE 39, PHASE II

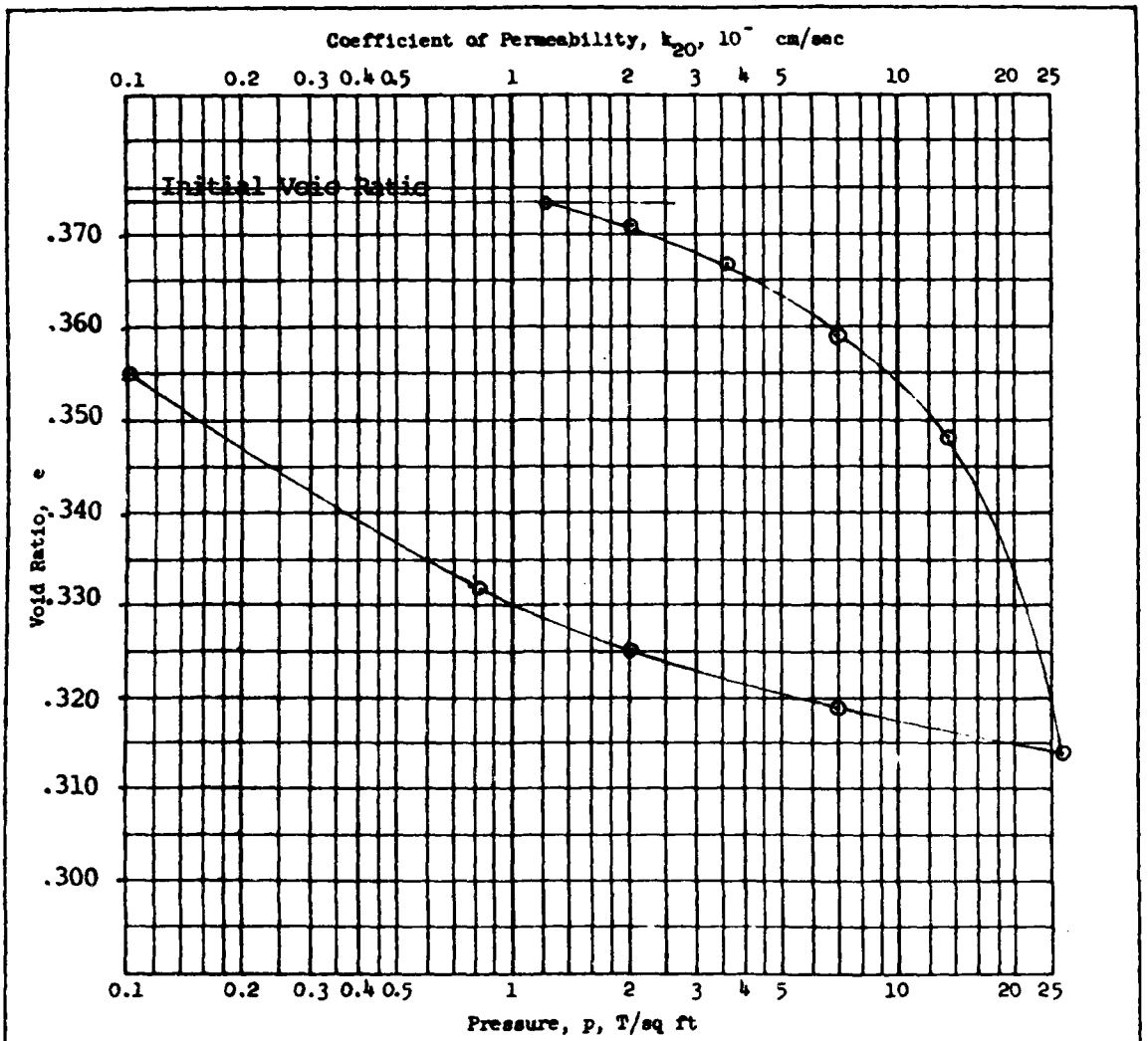


PROJECT	SEVEN OAKS DAM - IMPERVIOUS CORE			
AREA	Div.No 100045 & 46 FS No. 17 & 18 (Phase II)			
BORING NO. CBT-86-34	SAMPLE NO. Blend 34	DEPTH EL 0.75'-5.0'	DATE July 1987	

ENG FORM 2088 1 MAY 63 PREVIOUS EDITIONS ARE OBSOLETE.

CONSOLIDATION TEST—TIME CURVES (TRANSLUCENT)

PLATE 40, PHASE II

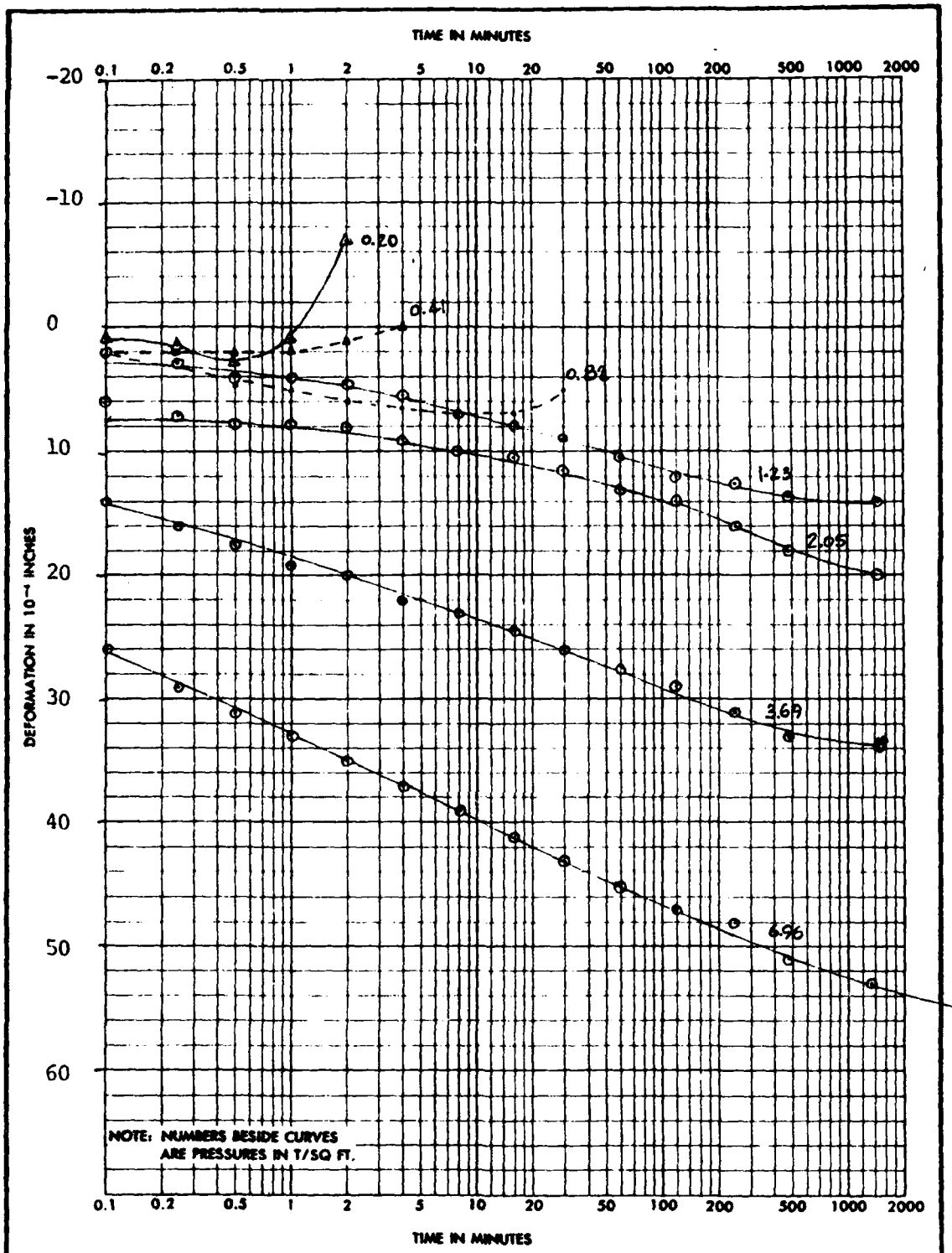


Type of Specimen: Remolded			Before Test			After Test	
Diam 4.4 in.	Ht 1.0 in.		Water Content, w_o	9.0 %		w_f	14.1 %
Overburden Pressure, Po T/sq ft			Void Ratio, e_o	0.373		e_f	0.381
Preconsol. Pressure, P_c T/sq ft			Saturation, S_o	65 %		S_f	100 %
Compression Index, C_c			Dry Density, γ_d	122.7 lb/ft^3			
Classification Sandy Clay (CL)			k_{20} at e_o =	× 10^{-} cm/sec			
LL 27	G_s 2.70		Project SEVEN OAKS DAM				
PL 16	D_{10}		Impervious Core (PHASE II)				
Remarks Remolded to 101% max. density @ opt - 3.7% water content. Div.No.100045&46			Area FS No. 17 & 18				
Max. density=121.0 Pcf, Opt= 12.7%			Boring No. CBT-86-34		Sample No. Blend 3A		
			Depth El 0.75' - 5.0'		Date July 1987		
			CONSOLIDATION TEST REPORT				

ENG FORM 1 MAY 63 2090 PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT)

PLATE 41, PHASE II

GPO 936-736



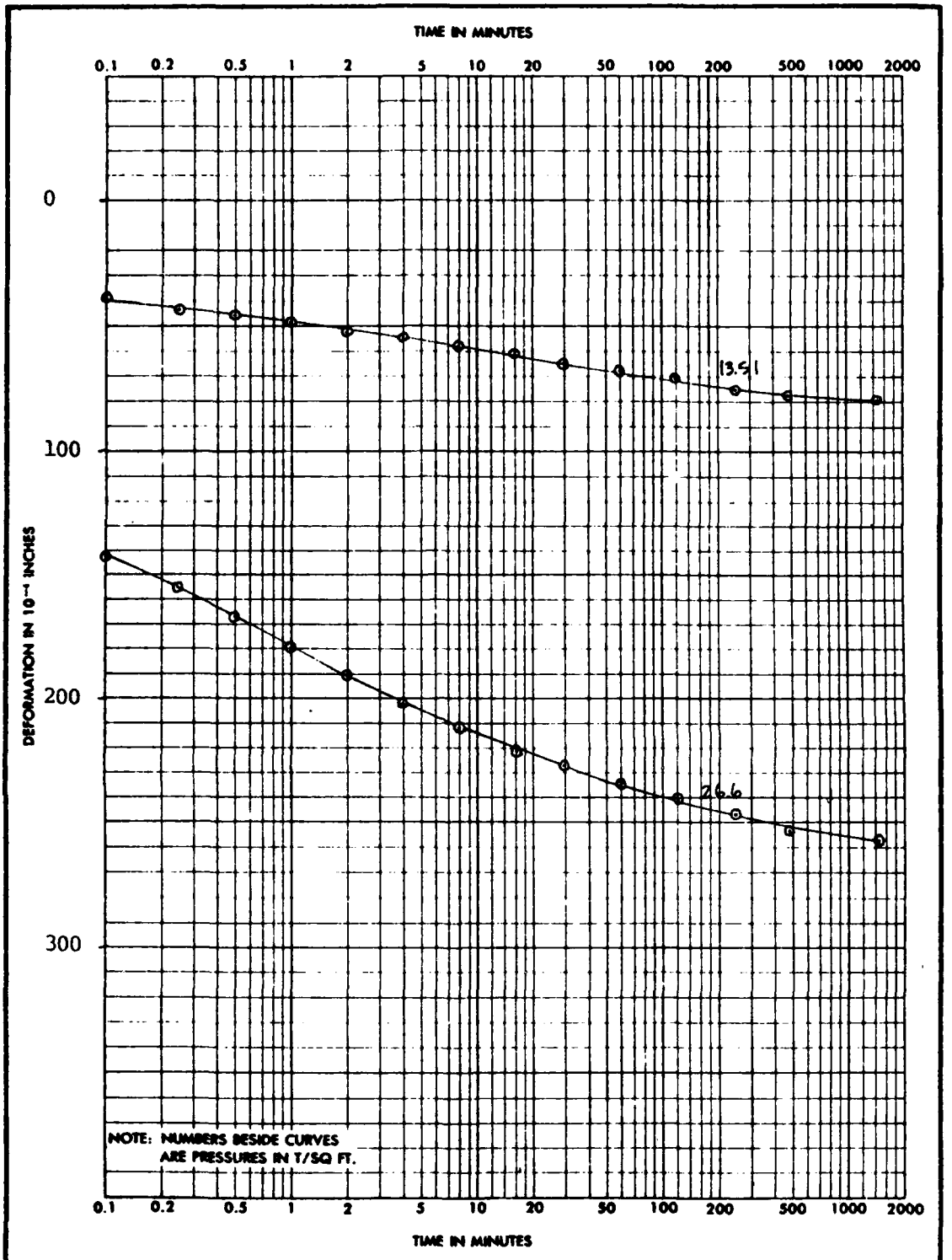
PROJECT	SEVEN OAKS DAM - Impervious Core			
AREA	Div.No.100045 & 46 FS No. 17 & 18 (PHASE II)			
BORING NO. CBT-86-34	SAMPLE NO. Blend 3A	DEPTH EL 0.75' - 5.0'	DATE July 1987	

ENG FORM 2088 1 MAY 63 PREVIOUS EDITIONS ARE OBSOLETE.

CONSOLIDATION TEST—TIME CURVES (TRANSLUCENT)

* GPO : 1964 O-715-969

PLATE 42, PHASE II

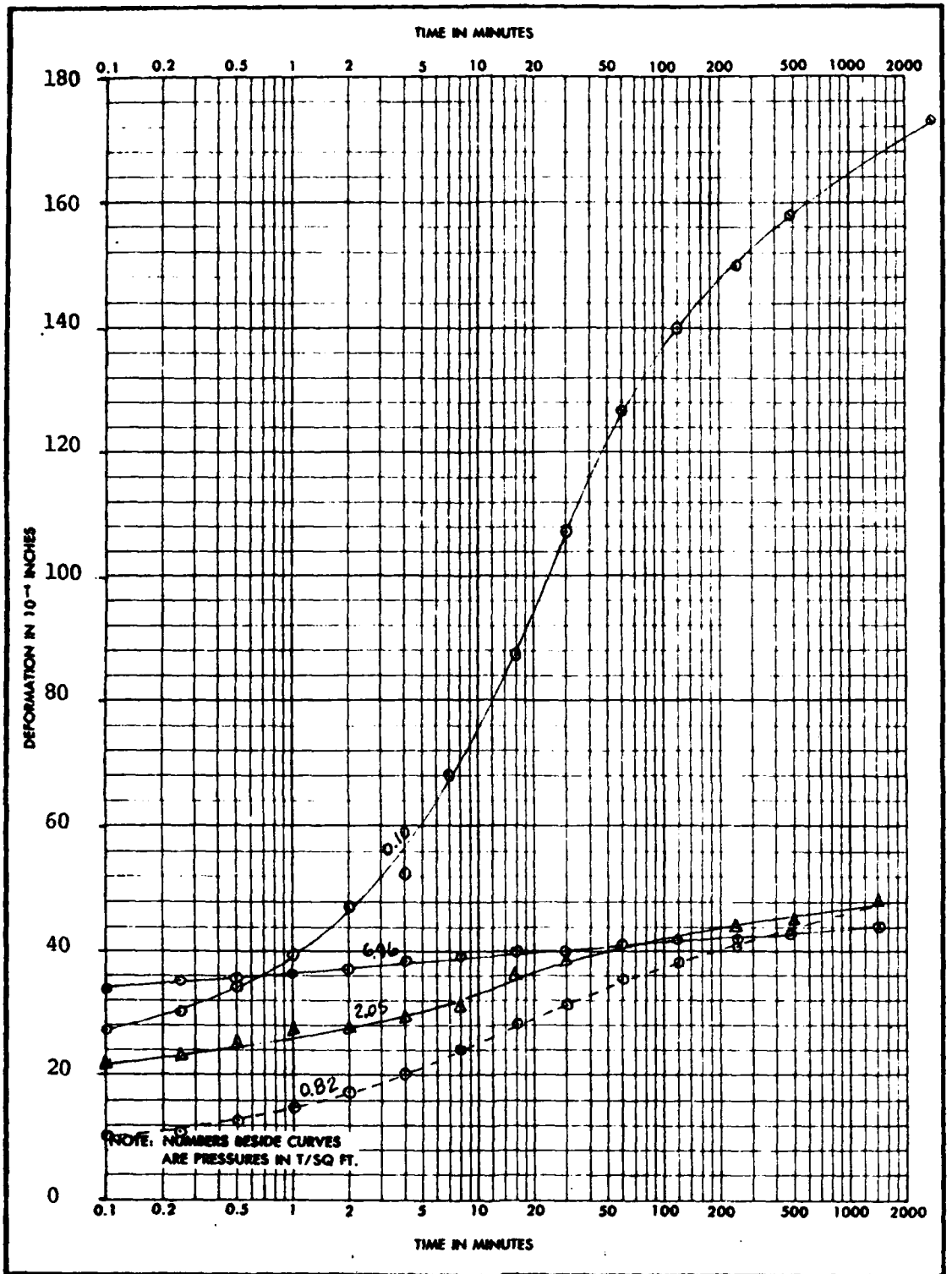


PROJECT	SEVEN OAKS DAM - Impervious Core			
AREA	Div.No.100045 & 46 FS No. 17 & 18 (PHASE II)			
BORING NO. CBT-86-34	SAMPLE NO. Blend 3A	DEPTH EL 0.5' - 5.0'	DATE July 1987	

ENG FORM 2088 1 MAY 63 PREVIOUS EDITIONS ARE OBSOLETE.

CONSOLIDATION TEST--TIME CURVES (TRANSLUCENT)

PLATE 43, PHASE II



PROJECT	SEVEN OAKS DAM - Impervious Core
AREA	Div.No.100045 & 46 FS No. 17 & 18 (PHASE II)
BORING NO.	CBT-86-34
SAMPLE NO.	Blend 3A
DEPTH EL	0.75' - 5.0'
DATE	July 1987

ENG FORM 2088 1 MAY 63 PREVIOUS EDITIONS ARE OBSOLETE.

CONSOLIDATION TEST—TIME CURVES (TRANSLUCENT)

* GPO : 1964 O-715-969

PLATE 44, PHASE II

U.S ARMY ENGINEER DIVISION LABORATORY - - SOUTH PACIFIC DIVISION

SOIL TEST RESULT SUMMARY

PROJECT: Upper Santa Ana River - Seven Oaks Dam (Impervious Core: Phase III) DATE: AUGUST 1987

Division Number	Hole Number	Field Sample No.	Depth or Elevation From	Depth or Elevation To	Laboratory Descriptive Classification	Mechanical Analysis - % Finer: 6	3	2	1.5	1	3/4	Gravel 1/2	3/8	#4	#10	Sand #40	#60	#100	Fines #200	Liquid Limit	Plasticity Index	Field Moist. %
100029	CBT 86-30 Blend 1B	1	0.75	5.0	Clayey Sand (SC)									100	92	64	54	46	36	35	15	
100030	CBT 86-30 Blend 2B	2	5.0	11.0	Clayey Sand (SC)			100	99	99	99	99	98	97	92	59	47	36	25	33	14	
100045 100046	CBT 86-34 Blend 3B	17 & 18	0.75	5.0	Sandy Clay (CL)									100	98	78	70	63	51	30	13	
100047	CBT 86-34 Blend 5B	19	27.5	32.0	Sandy Clay (CL)									100	97	80	73	65	53	30	14	
100050	CBT 86-38 Blend 6B	22	7.0	12.0	Clayey Sand (SC)	100	99	99	99	99	98	98	97	95	85	50	37	27	17	29	10	

SPD FORM 66A

PLATE 45, PHASE III

U.S ARMY ENGINEER DIVISION LABORATORY - - SOUTH PACIFIC DIVISION

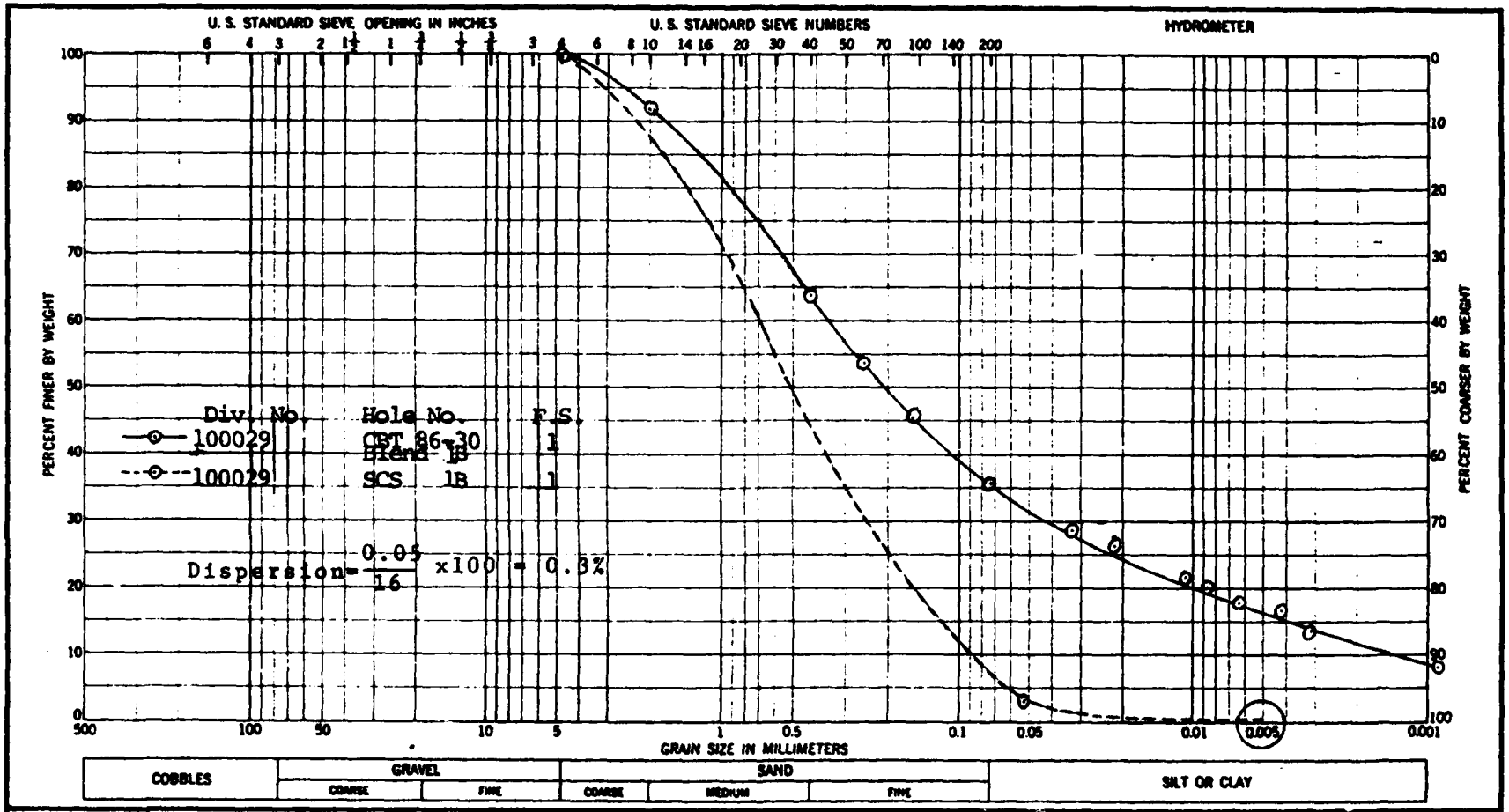
SOIL TEST RESULT SUMMARY

PROJECT: Upper Santa Ana River - Seven Oaks Dam (Impervious Core: Phase III) DATE: AUGUST 1987

Division Number	Hole Number	Field Sample No.	Depth or Elevation From	Depth or Elevation To	Laboratory Descriptive Classification	Mechanical Analysis - % Finer: 6	3	2	1.5	1	3/4	Gravel 1/2	3/8	#4	#10	Sand #40	#60	#100	Fines #200	Liquid Limit	Plasticity Index	Field Moist. %
100029	SCS 1B	1	0.75	5.0										100								
100030	SCS 2B	2	5.0	11.0				100 99		99 99		99	98	97								
100045 100046	SCS 3B	17 & 18	0.75	5.0										100								
100047	SCS 5B	19	27.5	32.0										100								
100050	SCS 6B	22	7.0	12.0		100 99		99 99		99 98		98	97	95								

SPD FORM 66A

PLATE 46 , PHASE III



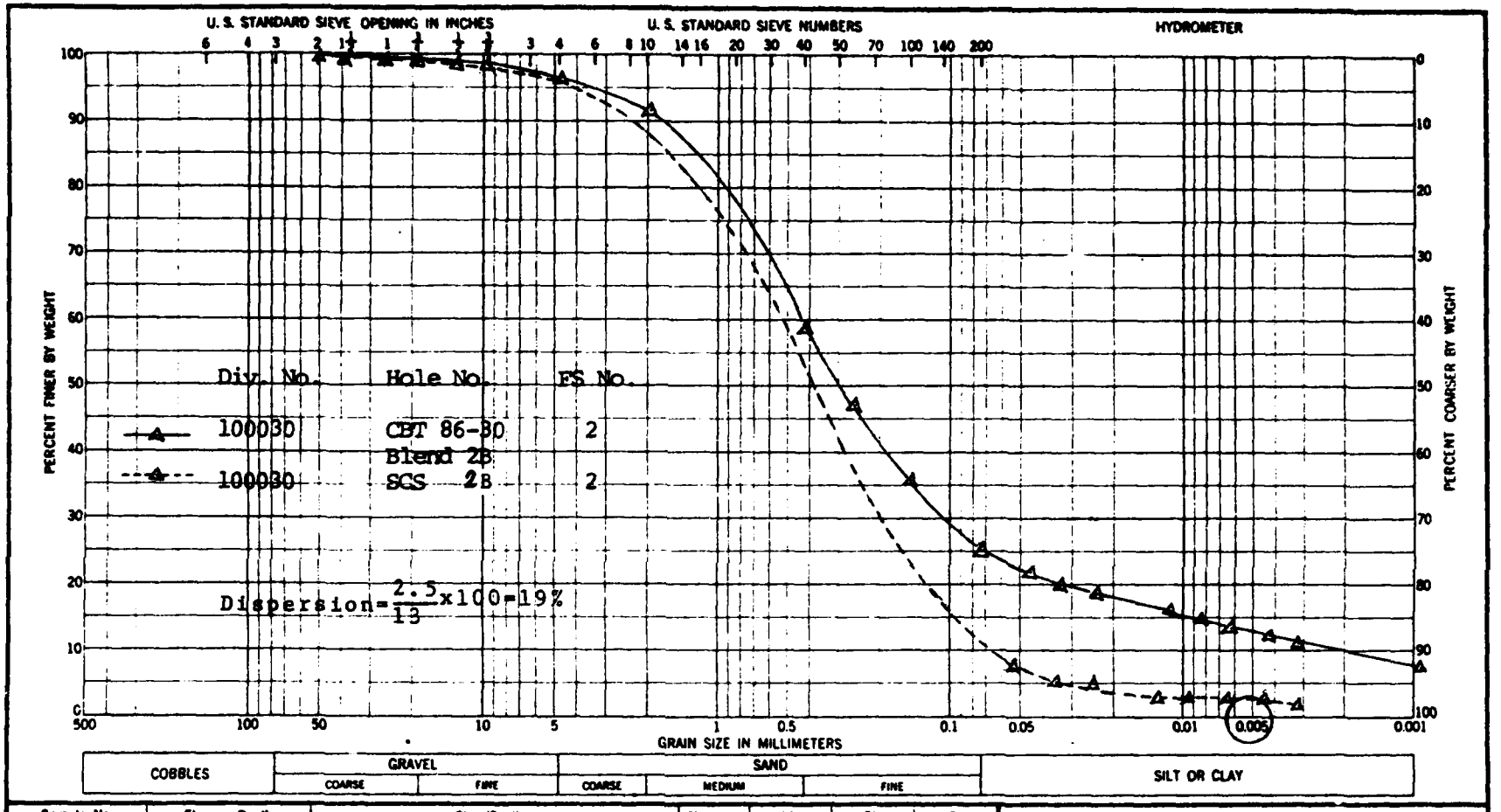
Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
1	0.75-5.0	Clayey Sand(SC)		35	20	15

Project: UPPER SANTA ANA RIVER
Seven Oaks Dam
Area: Impervious Core (Phase III)
Boring No.
Date: August 1987

GRADATION CURVES

ENG FORM 1 MAY 63 2087

PLATE 47, PHASE III



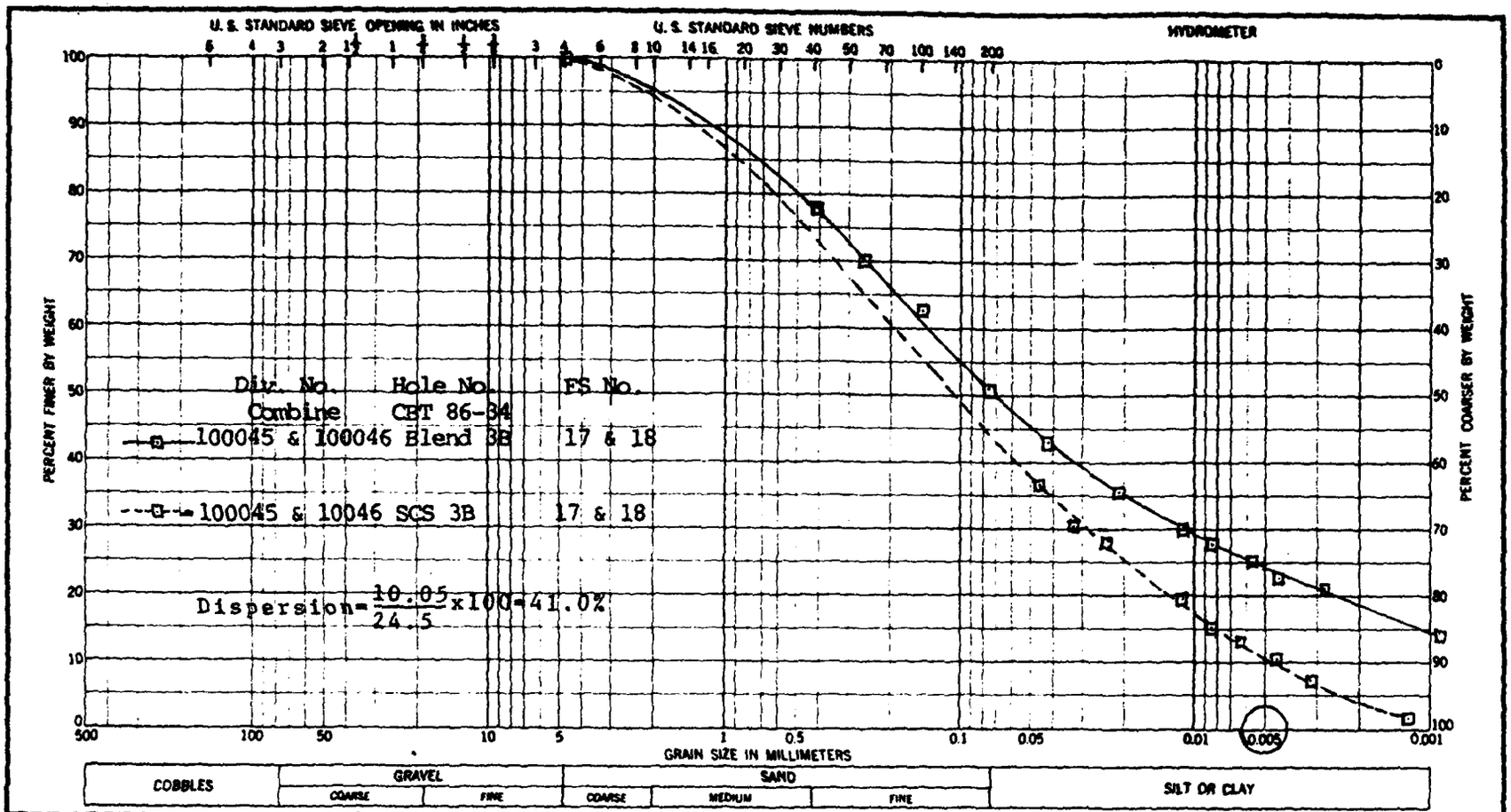
Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
2	5.0 - 11.0	Clayey Sand(SC)		33	19	14

GRADATION CURVES

Project UPPER SANTA ANA RIVER
SEVEN OAKS DAM
Area Impervious Core (Phase III)
Boring No.
Date August 1987

PLATE 48, PHASE III

ENG FORM 1 MAY 63 2087



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
17 & 18	0.75-5.0	Sandy Clay (CL)		30	17	13

GRADATION CURVES

Project UPPER SANTA ANA RIVER
SEVEN OAKS DAM

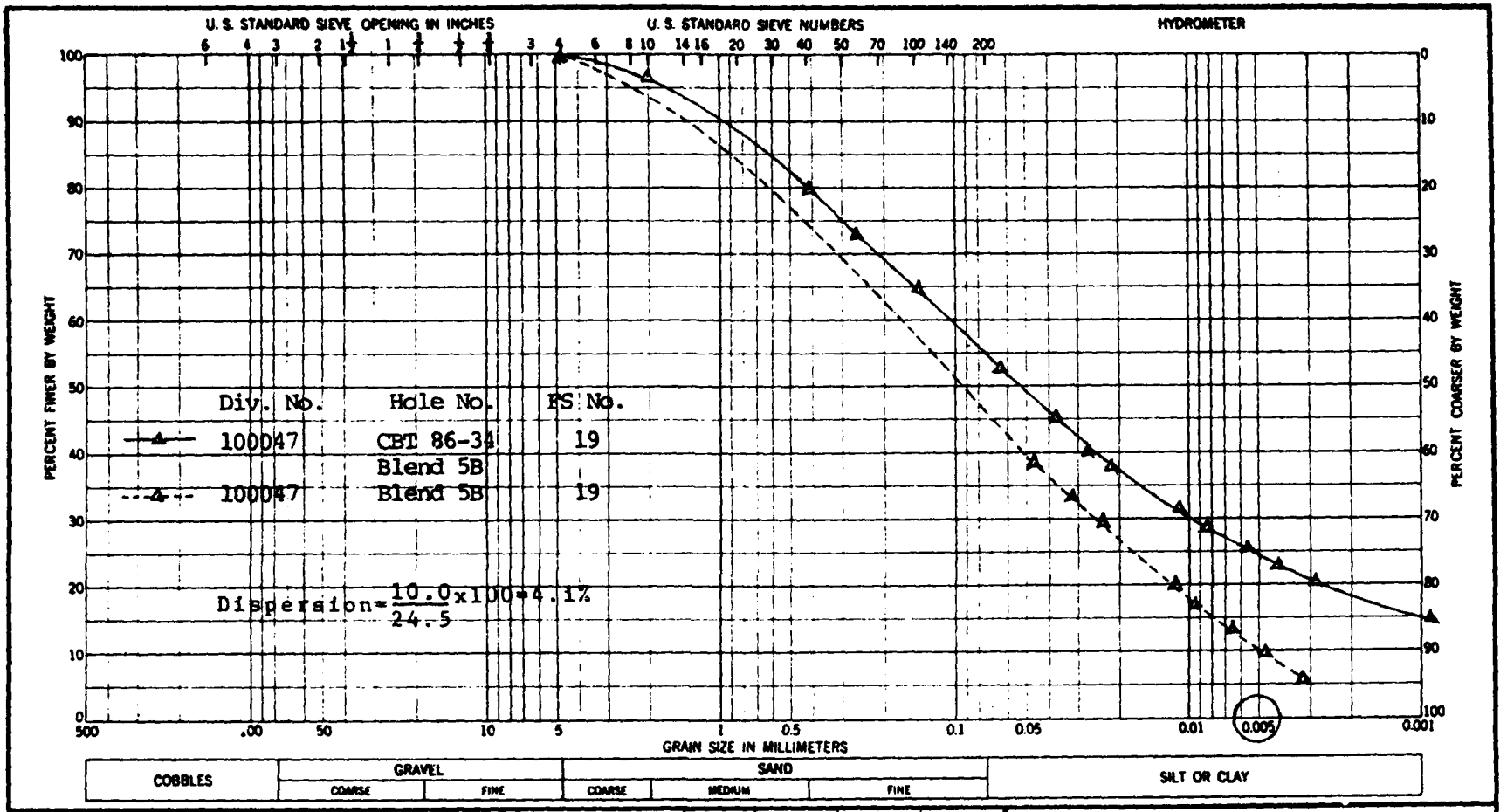
Area Impervious Core (Phase III)

Boring No.

Date August 1987

PLATE 49, PHASE III

ENG FORM 1 MAY 63 2087



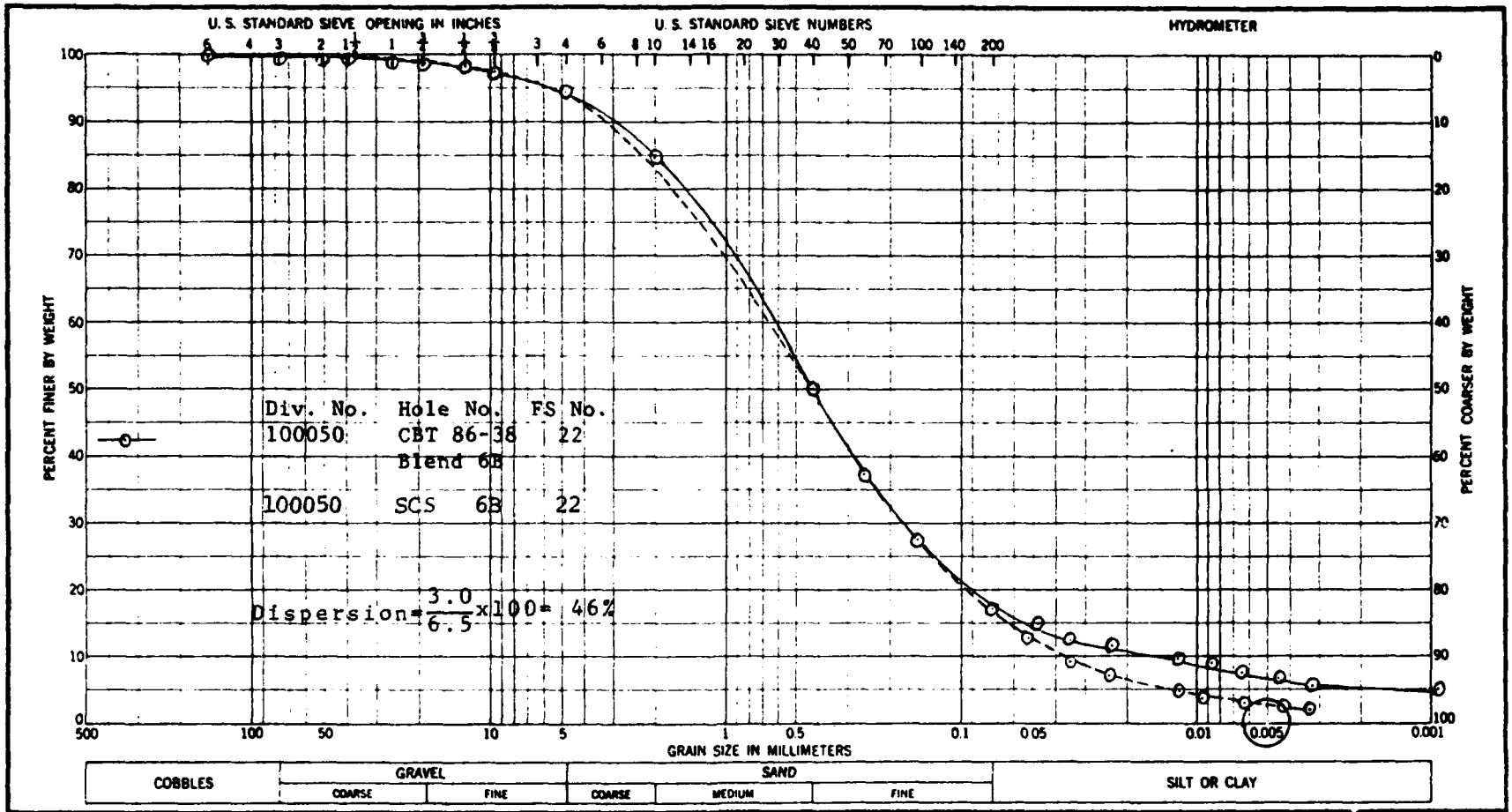
Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
19	27.5-32.0	Sandy Clay(CL)		30	20	14

GRADATION CURVES

Project UPPER SANTA ANA RIVER
SEVEN OAKS DAM
Area Impervious Core (Phase III)
Boring No. August 1987
Date

PLATE 50 , PHASE III

ENG FORM 1 MAY 63 2087



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
22	7.0-12.0	Clayey Sand (SC)		29	19	10

GRADATION CURVES

Project: Upper Santa Ana River
Seven Oaks Dam
Area: Impervious Core (Phase III)
Boring No.
Date: August 1987

PLATE 51, PHASE III

ENG FORM 1 MAY 63 2087

SPECIFIC GRAVITY
UPPER SANTA ANA - SEVEN OAKS DAM
IMPERVIOUS CORE (PHASE III)

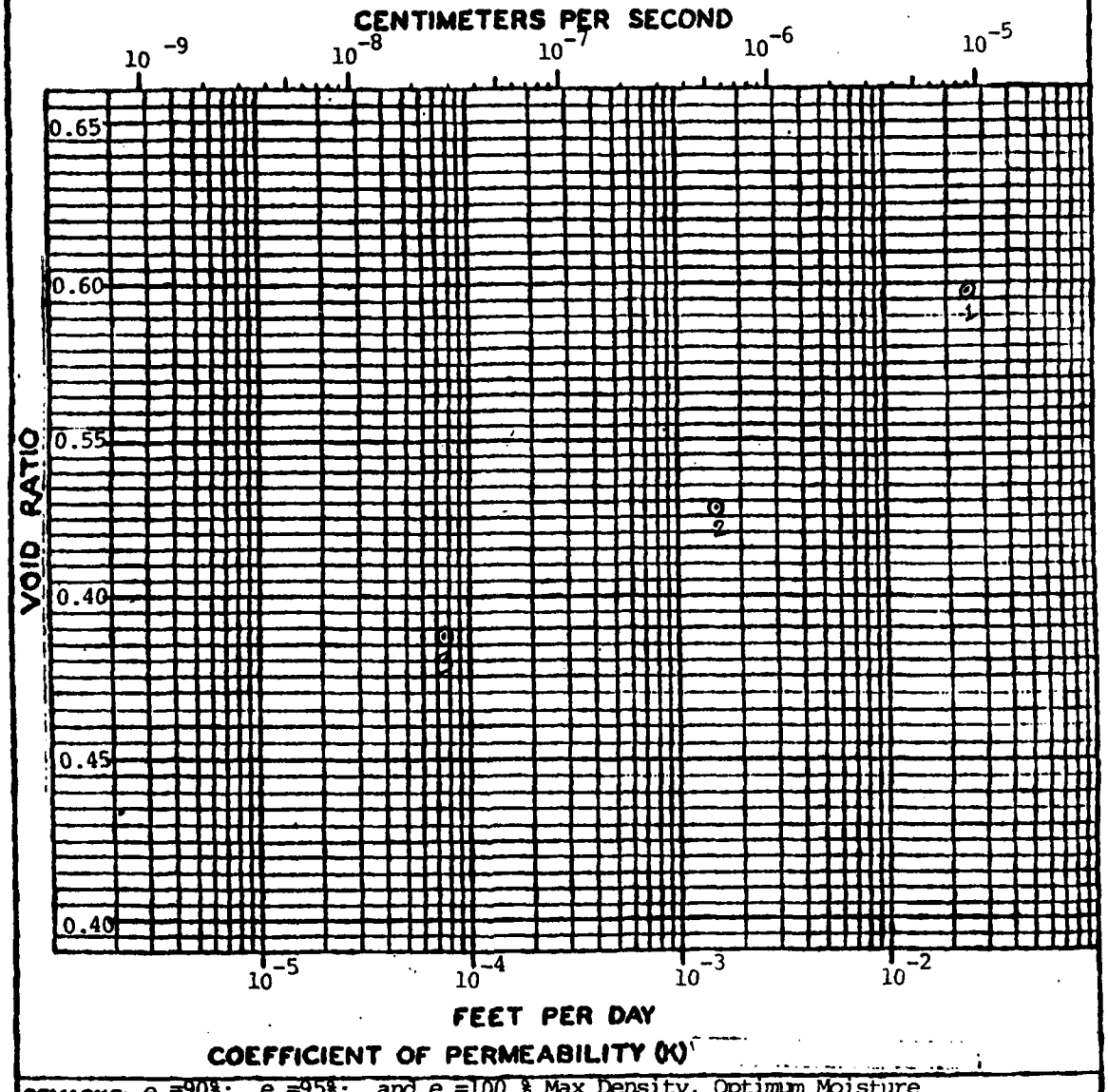
July 1987

Div. No.	Hole No.	F.S.	Blend	Elevation (ft)	Specific Gravity (-4)
100029	CBT 86-30	1	1B	0.75-5	2.69
100030	CBT 86-30	2	2B	5-11	2.70
100045-46	CBT 86-34	17 & 18	3B	0.75-5	2.71
"	"	"	4B	0.75-5	2.71
100047	CBT 86-34	19	5B	27.5-32	2.71
100050	CBT 86-38	22	6B	7-12	2.71

CORPS OF ENGINEERS — U. S. ARMY

SOUTH PACIFIC DIVISION LABORATORY

PERMEABILITY



REMARKS: e_1=90%; e_2=95%; and e_3=100 % Max Density, Optimum Moisture

Point	SPECIMEN DIAM. IN.	HT. IN.	MAX. PARTICLE	CONDITION
1	5.89	6.00	No. 4	Undisturbed.
2	5.80	5.91		
3	5.75	5.86		

DISTRICT: Los Angeles

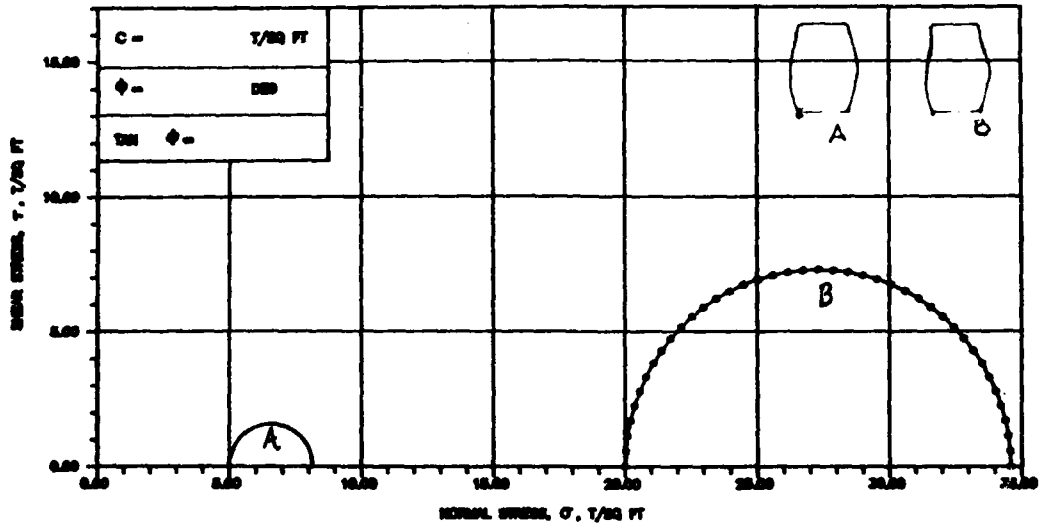
PROJECT: SEVEN OAKS DAM - Impervious Core, Phase III Blend 3B

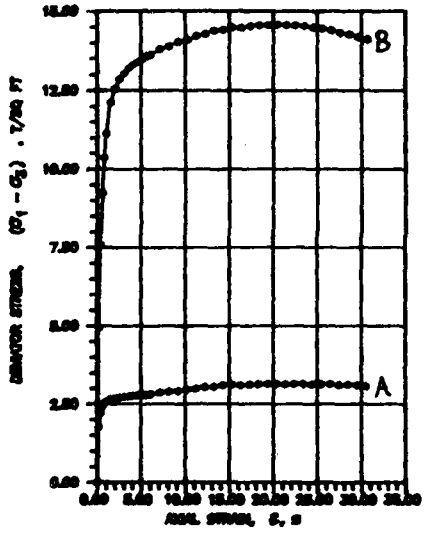
Point	DIV. NO.	HOLE NO.	F.S. NO.	DEPTH FROM	DEPTH TO
1	Combine				
2	100045 &	CBT-86-34	17 & 18	0.75	5
3	100046				

TESTED	COMPUTED	DRAWN	CHECKED
FN	FN	CRK	DW

SPD FORM 6 OCT 58 67 (CIVIL; REPLACES PSD 67, (CIVIL), 20 JULY 49, WHICH MAY BE USED.

PLATE 53 PHASE III





SPECIMEN NO.			A	B		
INITIAL	WATER CONTENT, %	w_o	16.0	12.8		
	DRY DENSITY LB/CU FT	γ_{d_o}	111.3	111.6		
	SATURATION, %	S_o	85	69		
	VOID RATIO	e_o	.502	.499		
BEFORE SHEAR	WATER CONTENT, %	w_c	15.4	12.4		
	DRY DENSITY LB/CU FT	γ_d	118.3	125.9		
	SATURATION, %	S_o	100	100		
	VOID RATIO	e_o	.414	.332		
	PORE BACK PRESSURE, T/SQ FT	u_o	6.50	4.30		
CONFINING PRESSURE, T/SQ FT		σ_3	5.0	20.7		
DEVIATOR STRESS, T/SQ FT	$(\sigma_1 - \sigma_3)_{MAX}$		3.15	14.45		
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	315	389		
ULTIMATE DEVIATOR STRESS, T/SQ FT	$(\sigma_1 - \sigma_3)_{ULT}$		-	-		
INITIAL DIAMETER, IN.		D_o	2.80	2.80		
INITIAL HEIGHT, IN.		H_o	6.45	6.45		

CONTROLLED STRAIN TEST: Yes

DESCRIPTION OF SPECIMEN: Sandy Clay (CL)

LL 30	PL 17	PI 13	G_s 2.68
REMARKS: Div.No.100045&100046			
Remolded to 98% max.density			
@16% & 13% water content.			
max. density=14.0%			
Opt. water=14.0%			
3% Bentonite by Wt.			

TYPE OF SPECIMEN: Remolded

TYPE OF TEST: "R"

PROJECT: Seven Oaks Dam-ImperviousCore (Phase III)

BORING NO. CBT 86-34

SAMPLE NO. Blend 3B

LABORATORY SPD

DATE Nov.1987

TRIAXIAL COMPRESSION TEST REPORT

USD FORM ED. 2089 REV. JUNE 1970

PLATE 54, PHASEIII

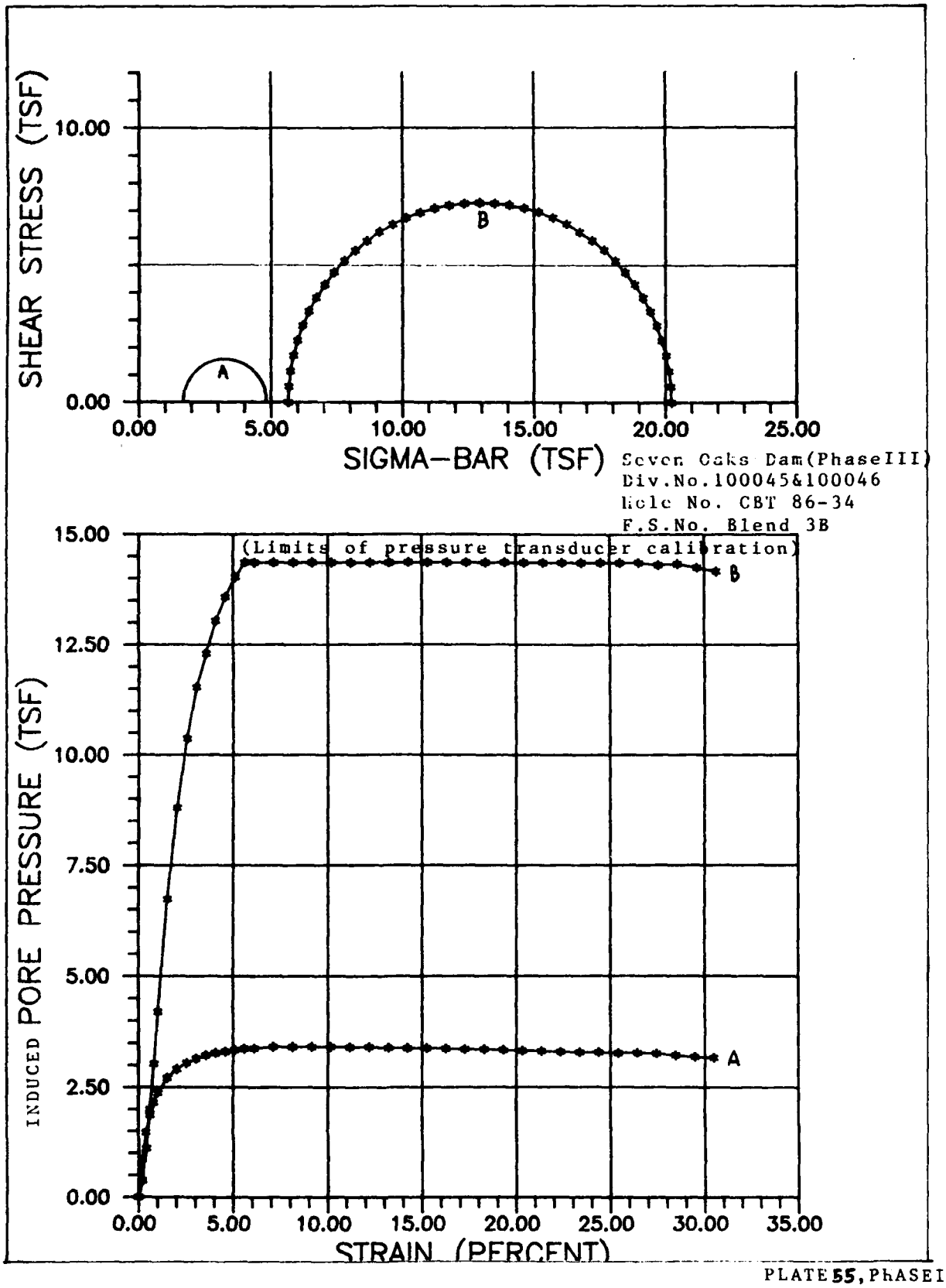
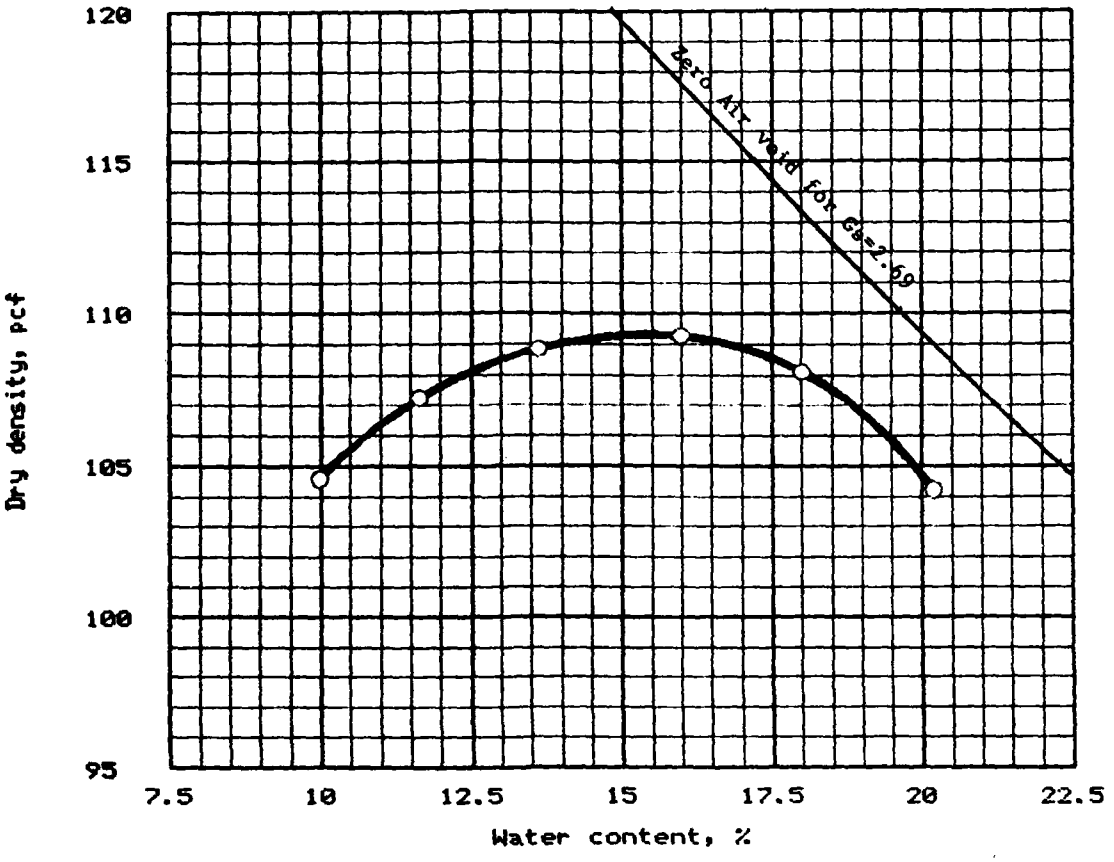


PLATE 55, PhASEIII

COMPACTION TEST REPORT



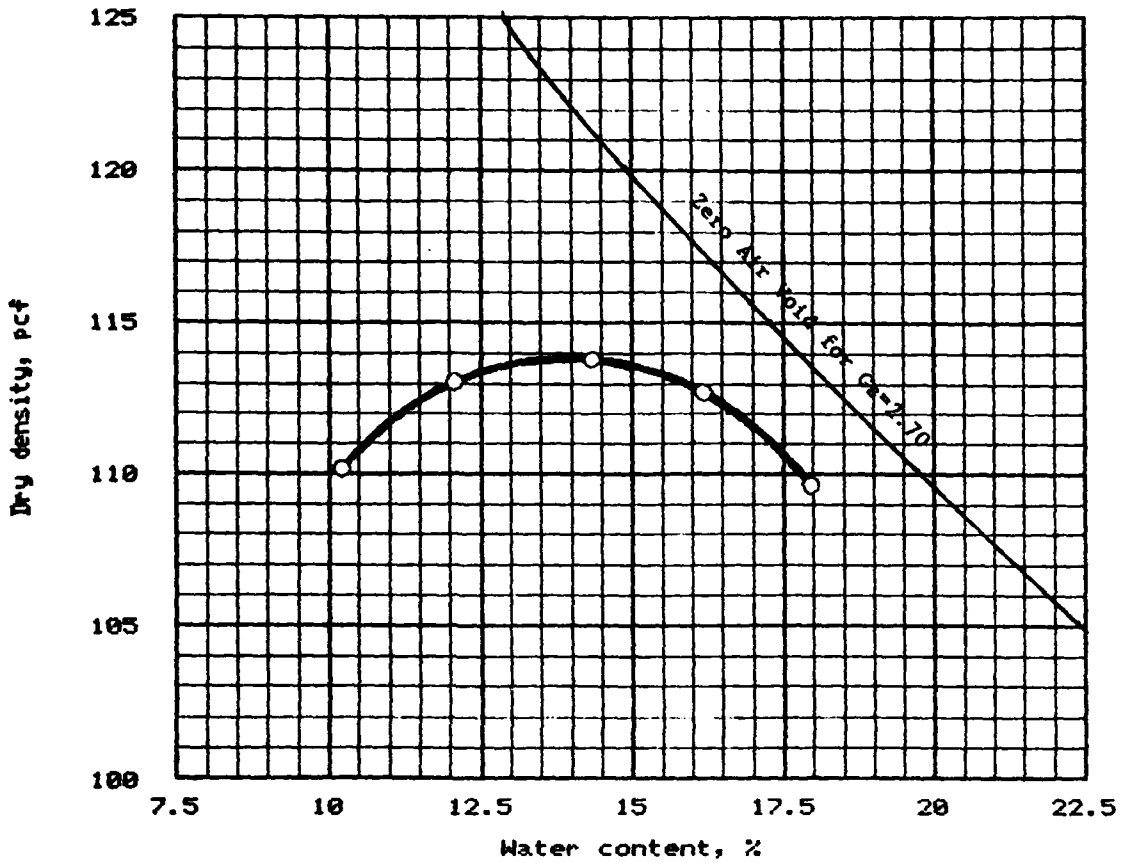
"Standard" Proctor, ASTM D 698-78, Method A

Elev/ Depth	USCS Classification	Nat. Moist.	Sp.G.	LL	PI	% >	% <
0.75-5.0	Clayey Sand (SC)		2.69	35	15	No. 4	No. 200

TEST RESULTS	MATERIAL DESCRIPTION
Optimum moisture = 15.4 % Maximum dry density = 109.3 pcf	Remolded
Division No.: 100029 Field Sample No. 1 Project: SEVEN OAKS DAM Location: IMPERVIOUS CORE Hole No. CBT-86-30 Date: 7-15-1987	Remarks: BARREL 1 BENTONITE BLEND 1-B (Phase III)
COMPACTION TEST REPORT CORPS OF ENGINEERS - SPDL	Plate No. 56

PHASE III

COMPACTION TEST REPORT



"Standard" Proctor, ASTM D 698-78, Method A

Elev/ Depth	USCS Classification	Nat. Moist.	Sp.G.	LL	PI	% >	% <
5.0-11.0	Clayey Sand (SC)		2.70	33	14	No. 4	No. 200

TEST RESULTS	MATERIAL DESCRIPTION
Optimum moisture = 14.0 % Maximum dry density = 113.8 pcf	Remolded

Division No.: 100030 F.Sample No. 2
Project: SEVEN OAKS DAM
Hole No.: CBT-86-30
Location: Impervious Core
Date: 7-15-1987

Remarks:
BENTONITE BLEND 2-B

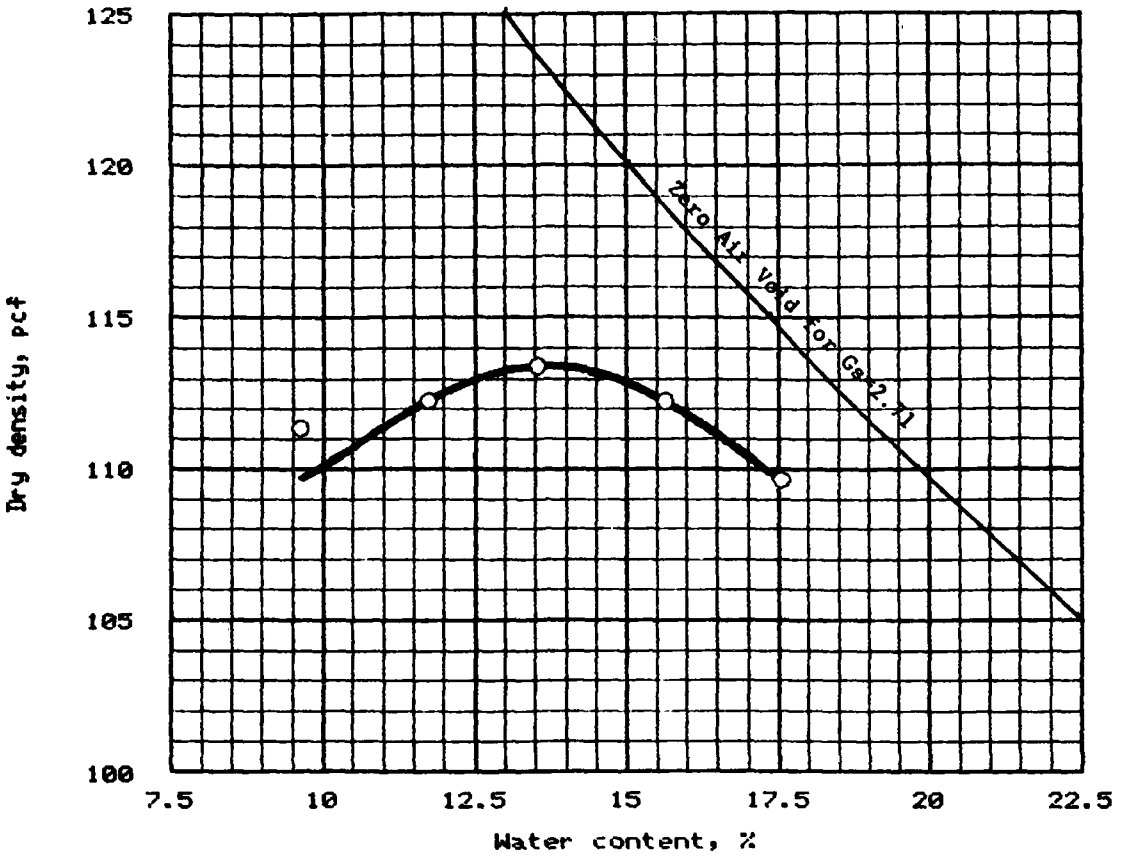
(Phase III)

COMPACTION TEST REPORT
CORPS OF ENGINEERS - SPDL

Plate No. 57

PHASE III

COMPACTION TEST REPORT



"Standard" Proctor, ASTM D 698-78, Method A

Elev/ Depth	USCS Classification	Nat. Moist.	Sp.G.	LL	PI	% >	% <
0.75-5.0	Sandy Clay (CL)		2.71	30	13	No. 4	No. 200

TEST RESULTS	MATERIAL DESCRIPTION
Optimum moisture = 13.7 % Maximum dry density = 113.4 pcf	Remolded

Division No.: 100045, 46 Field Sample No. 17 & 18
Project: SEVEN OAKS DAM
Location: IMPERVIOUS CORE
Hole No. CBT-86-34
Date: 7-15-1987

Remarks:
BARRELS 17&18
BENTONITE BLEND 3-B
(Phase III)

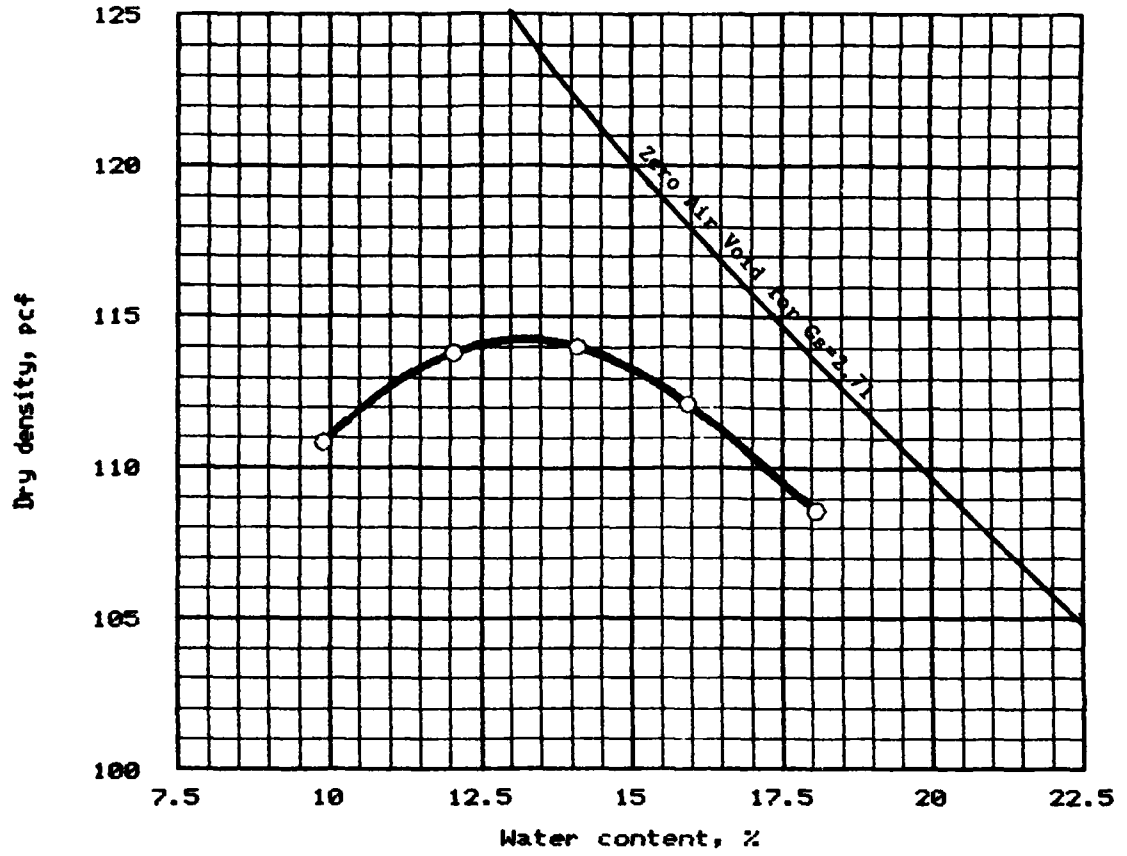
COMPACTION TEST REPORT

CORPS OF ENGINEERS - SPDL

Plate No. 58

PHASE III

COMPACTION TEST REPORT



"Standard" Proctor, ASTM D 698-78, Method A

Elev/ Depth	USCS Classification	Nat. Moist.	Sp.G.	LL	PI	% >	% <
0.75-5.0	Sandy Clay (CL)		2.71	30	13	No. 4	No. 200

TEST RESULTS	MATERIAL DESCRIPTION
Optimum moisture = 13.2 % Maximum dry density = 114.3 pcf	Remolded

Division No.: 100045,46 Field Sample No. 17 & 18
Project: SEVEN OAKS DAM
Location: IMPERVIOUS CORE
Hole No. CBT-86-34
Date: 7-14-1987

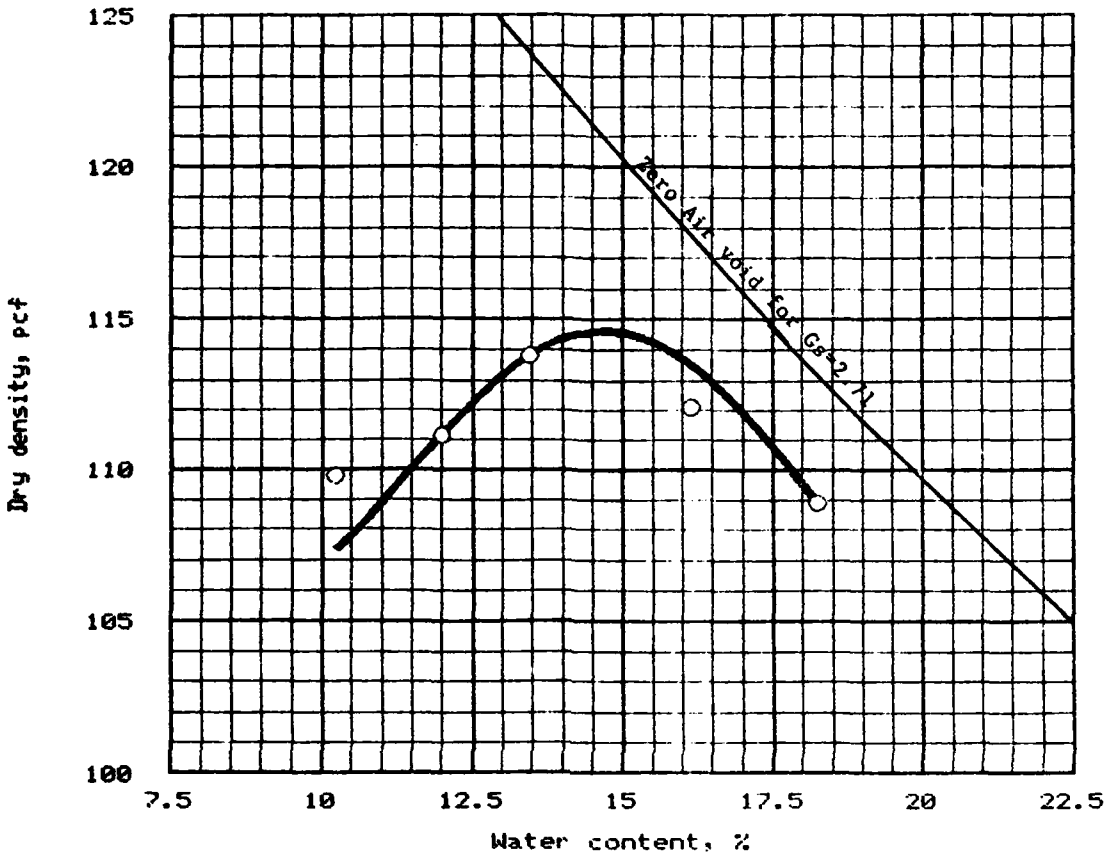
Remarks:
BARRELS 17&18
BENTONITE BLEND 4-B
(Phase III)

COMPACTION TEST REPORT
CORPS OF ENGINEERS - SADL

Plate No. 59

PHASE III

COMPACTION TEST REPORT



"Standard" Proctor, ASTM D 698-78, Method A

Elev/ Depth	USCS Classification	Nat. Moist.	Sp.G.	LL	PI	% >	% <
27.5-32.0	Sandy Clay (CL)		2.71	30	14	No. 4	No. 200

TEST RESULTS	MATERIAL DESCRIPTION
Optimum moisture = 14.7 % Maximum dry density = 114.6 pcf	Remolded

Division No.: 100047 Field Sample No. 19
Project: SEVEN OAKS DAM
Location: IMPERVIOUS CORE
BORING NO. CBT-86-34
Date: 7-15-1987

Remarks:
BLEND 5-B
BARREL NO.19
(Phase III)

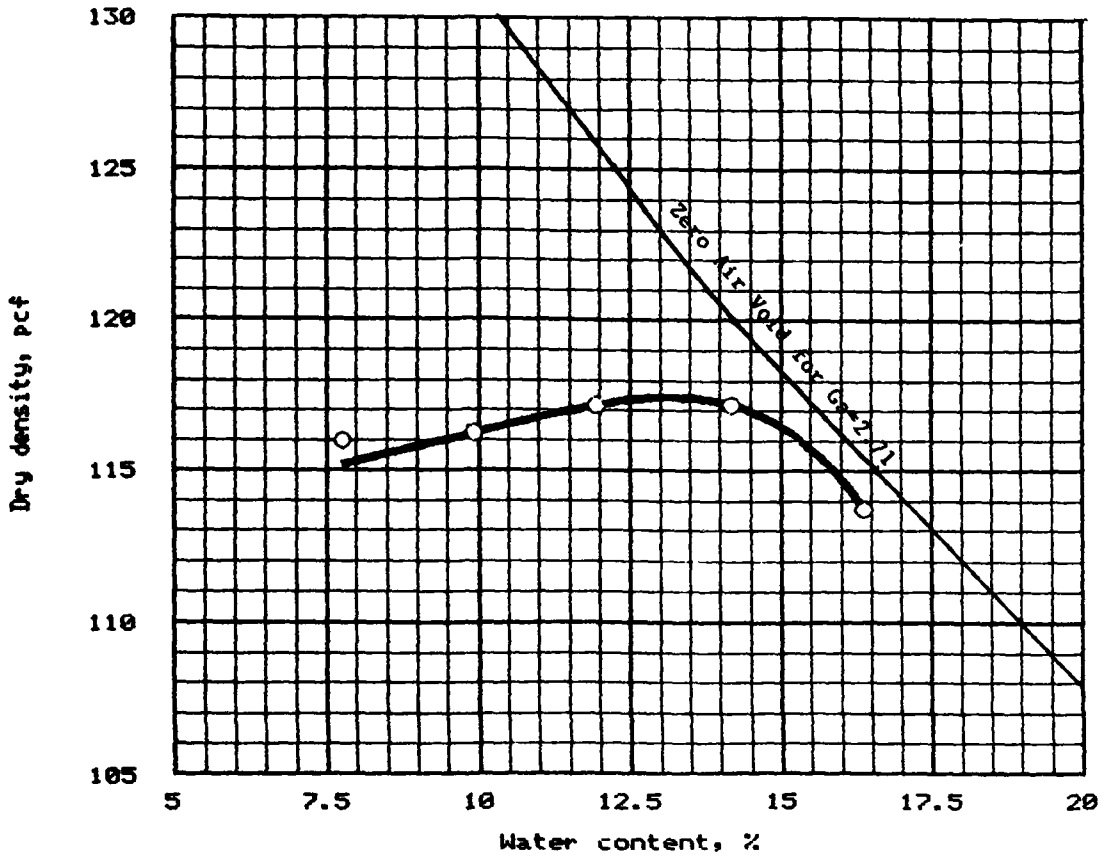
COMPACTION TEST REPORT

CORPS OF ENGINEERS - SPDL

Plate No. 60

PHASE III

COMPACTION TEST REPORT



"Standard" Proctor, ASTM D 698-78, Method A

Elev/ Depth	USCS Classification	Nat. Moist.	Sp.G.	LL	PI	% >	% <
7.0-12.0	Clayey Sand (SC)		2.71	29	10	No. 4	No. 200

TEST RESULTS	MATERIAL DESCRIPTION
Optimum moisture = 13.2 % Maximum dry density = 117.4 pcf	Remolded

Division No.: 100050 Field Sample No. 22

Project: SEVEN OAKS DAM

Location: IMPERVIOUS CORE

Hole No. CBT-86-38

Date: 7-15-1987

Remarks:

BARREL 22

BENTONITE BLEND 6-B

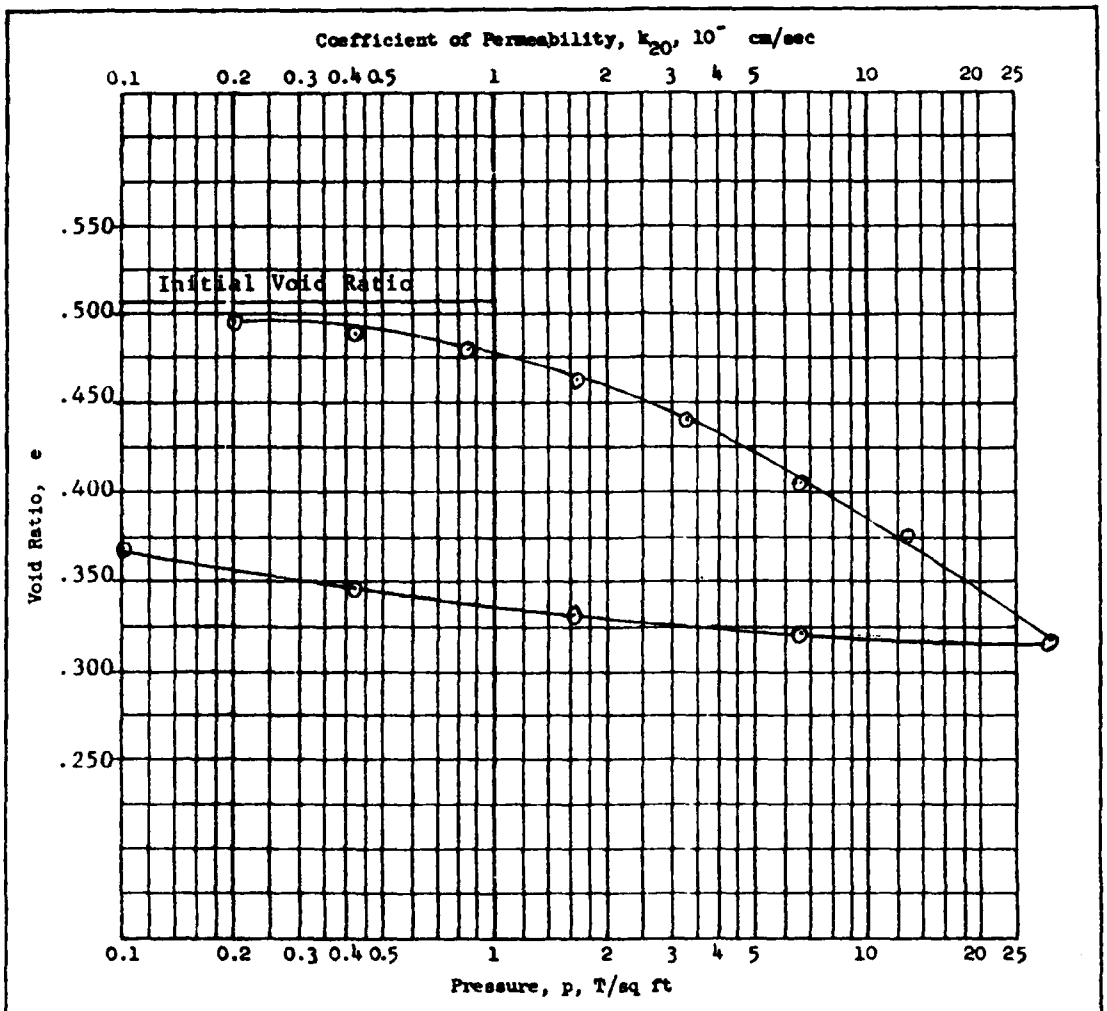
(Phase III)

COMPACTION TEST REPORT

CORPS OF ENGINEERS - SPOIL

Plate No. 61

PHASE III



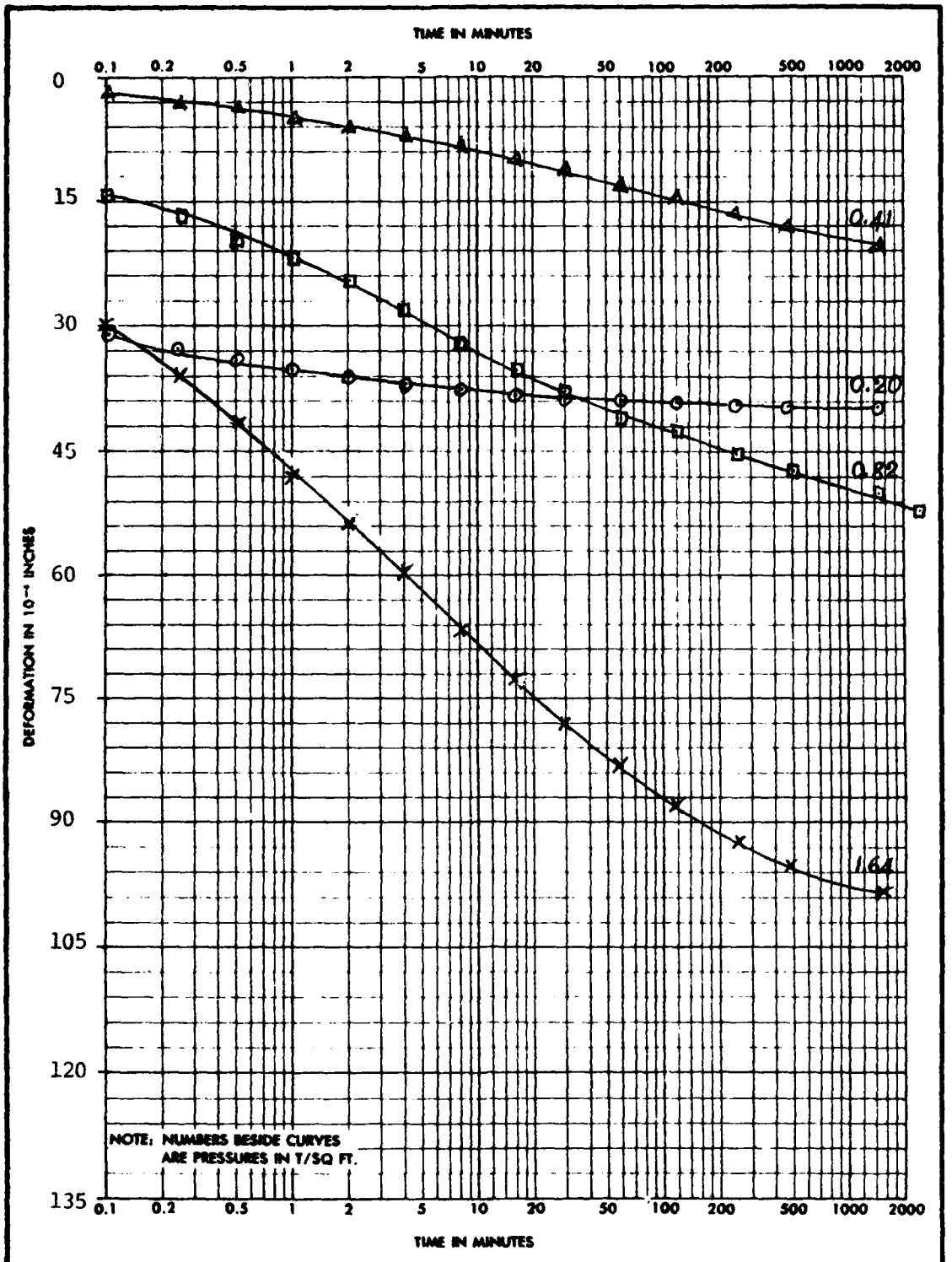
Type of Specimen Remolded			Before Test		After Test	
Diam 4.4 in.	Ht 1 in.		Water Content, w_o	14.3 %	w_f	14.5 %
Overburden Pressure, Po T/sq ft			Void Ratio, e_o	0.504	e_f	0.395
Preconsol. Pressure, Pc T/sq ft			Saturation, S_o	75.8 %	S_f	98.4 %
Compression Index, C_c			Dry Density, γ_d	111.2 lb/ft^3		
Classification Sandy Clay (CL)			k_{20} at e_o =	x 10^{-} cm/sec		
LL 30	G_s 2.71		Project SEVEN OAKS DAM - Impervious Core			
PL 17	D_{10}		Combine 100045 & 100046 (Blend 3B)			
Remarks			Area Phase III			
Remolded to 98% max. density			Boring No. CBT-86-34		Sample No. 17 & 18	
@ optimum			Depth El 0.75'-5'		Date July 1987	
			CONSOLIDATION TEST REPORT			

ENG FORM 2090 1 MAY 63 PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT)

GPO 936-736

n 3424

PLATE 62 - PHASE III

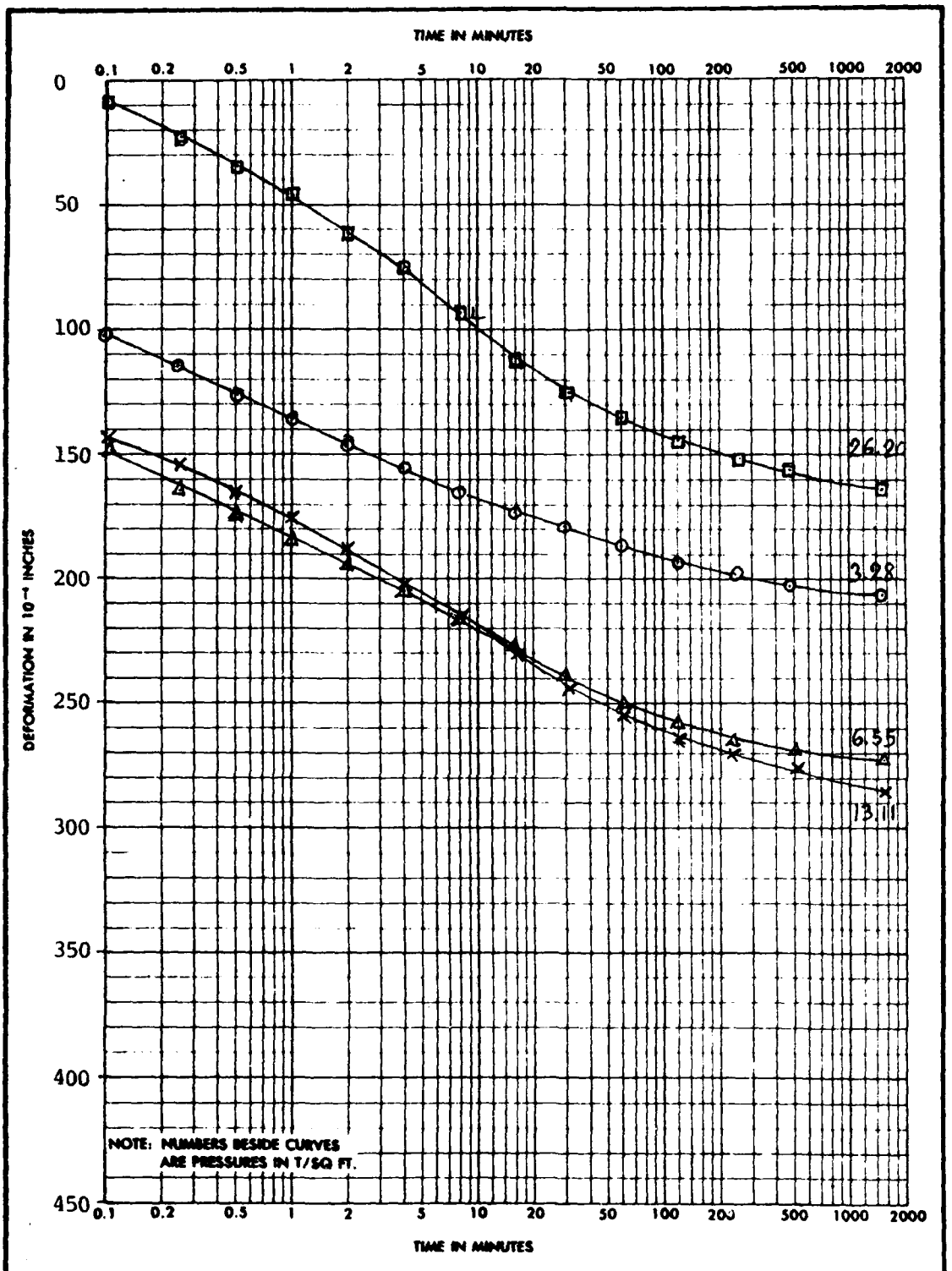


PROJECT	SEVEN OAKS DAM - Impervious Core			
AREA	Combine 100045 & 100046 (Blend 3B)			Phase III
BORING NO. CBT-86-34	SAMPLE NO. 17 & 18	DEPTH EL 0.75' - 5'		DATE July 1987

ENG FORM 2088 1 MAY 63 PREVIOUS EDITIONS ARE OBSOLETE.

CONSOLIDATION TEST—TIME CURVES (TRANSLUCENT)

PLATE 63, PHASE III

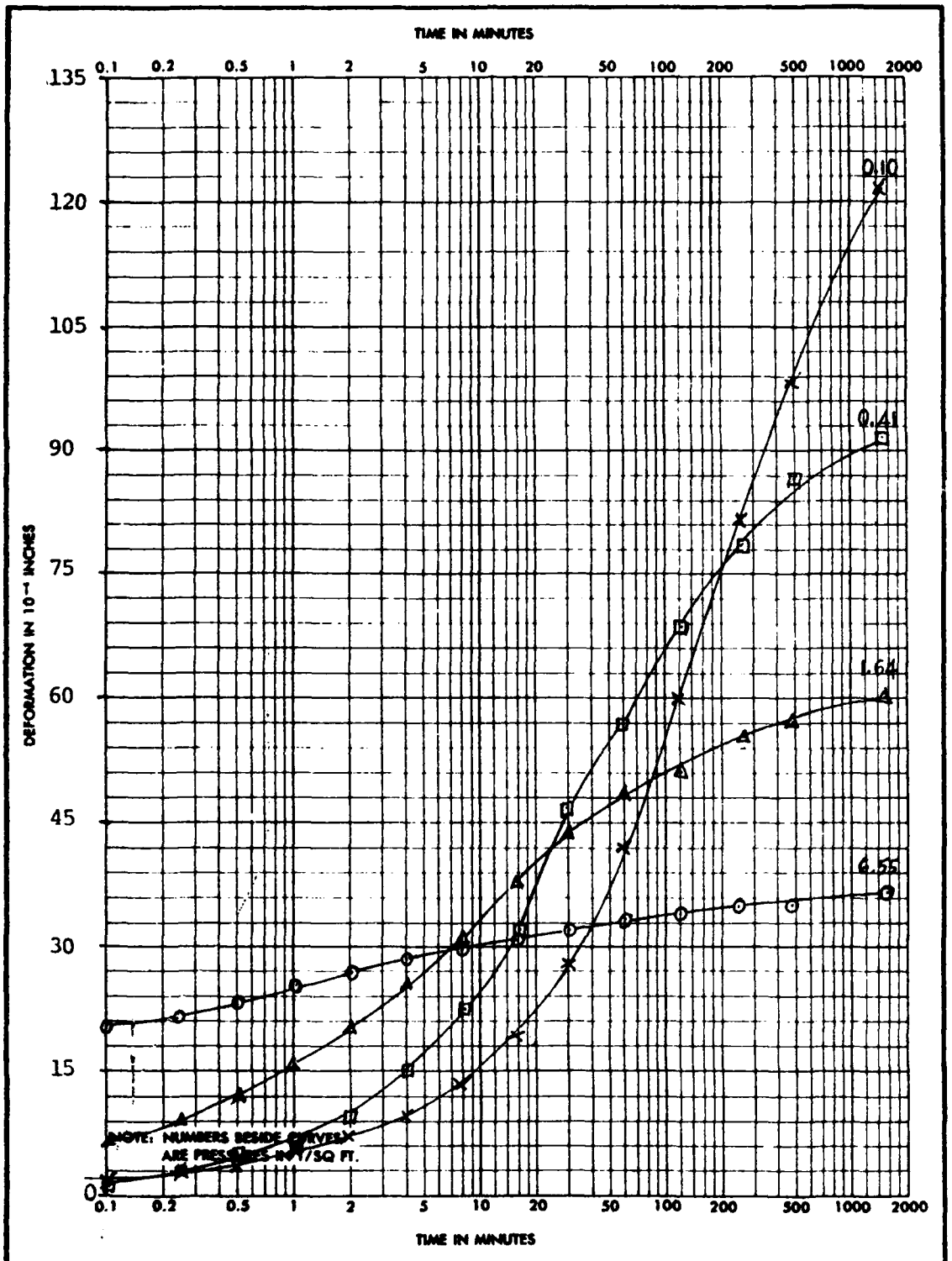


PROJECT	SEVEN OAKS DAM - Impervious Core
AREA	Combine 100045 & 100046 (Blend 3B) Phase III
BORING NO. CBT-86-34	SAMPLE NO. 17 & 18
DEPTH EL	0.75' - 5'
DATE	July 1987

ENG FORM 2088 1 MAY 63 PREVIOUS EDITIONS ARE OBSOLETE.

CONSOLIDATION TEST—TIME CURVES (TRANSLUCENT)

PLATE 64 , PHASE III



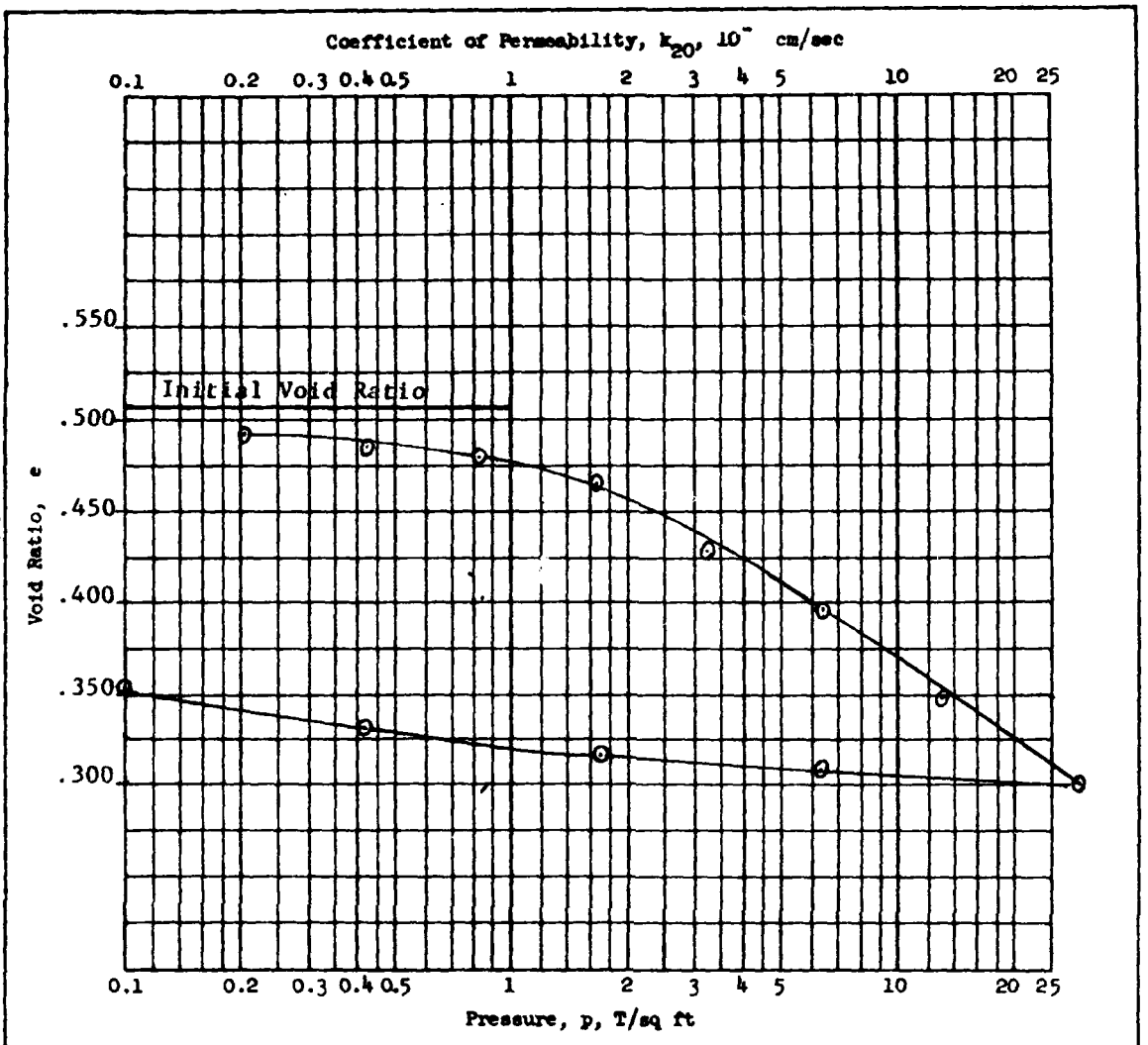
PROJECT	SEVEN OAKS DAM - Impervious Core			
AREA	Combine 100045 & 100046 (Blend 3B)			Phase III
BORING NO. CBT-86-34	SAMPLE NO. 17 & 18	DEPTH EL 0.75' - 5'	DATE July 1987	

ENG FORM 2088 1 MAY 63 PREVIOUS EDITIONS ARE OBSOLETE.

CONSOLIDATION TEST--TIME CURVES (TRANSLUCENT)

* GPO : 1964 OF—715-965

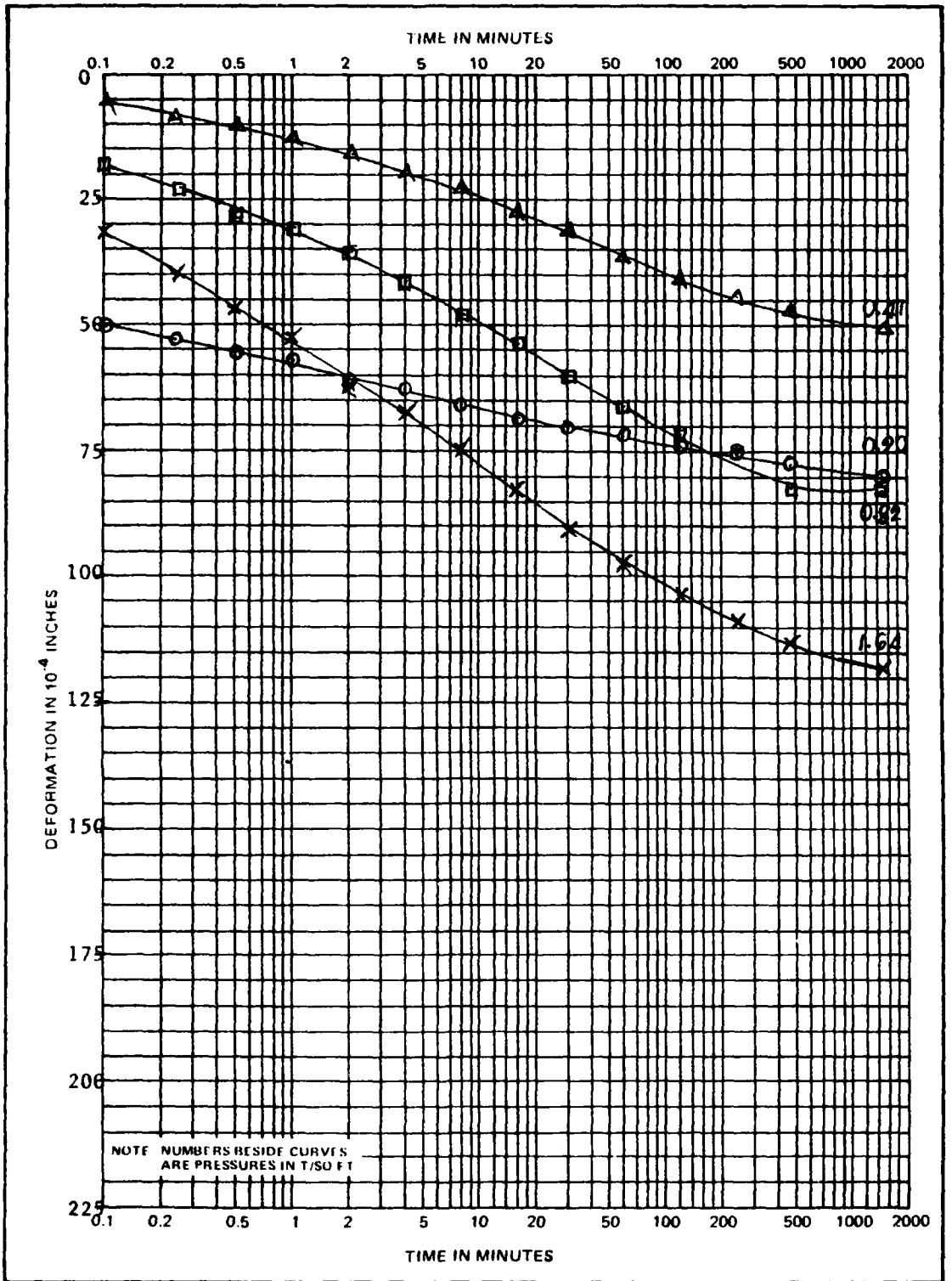
PLATE 65 , PHASE III



Type of Specimen Remolded		Before Test			After Test	
Diam 4.4 in.	Ht 1 in.	Water Content, w_o	16.2 %	w_f	13.77 %	
Overburden Pressure, p_o T/sq ft		Void Ratio, e_o	0.504	e_f	0.369	
Preconsol. Pressure, p_c T/sq ft		Saturation, S_o	86.2 %	S_f	100 %	
Compression Index, C_c		Dry Density, γ_d	111.2 lb/ft^3			
Classification Sandy Clay (CL)		k_{20} at e_o =	x 10^{-} cm/sec			
LL 30	G_s 2.71	Project SEVEN OAKS DAM - Impervious Core				
PL 17	D_{10}	Combine 100045 & 100046 (Blend 3B)				
Remarks		Area Phase III				
Remolded to 98% max. density		Boring No. CRT-86-34		Sample No. 17 & 18		
at opt. +2%		Depth El 0.75'-5		Date July 1987		
		CONSOLIDATION TEST REPORT				

ENG FORM 1 MAY 63 2090 PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT)

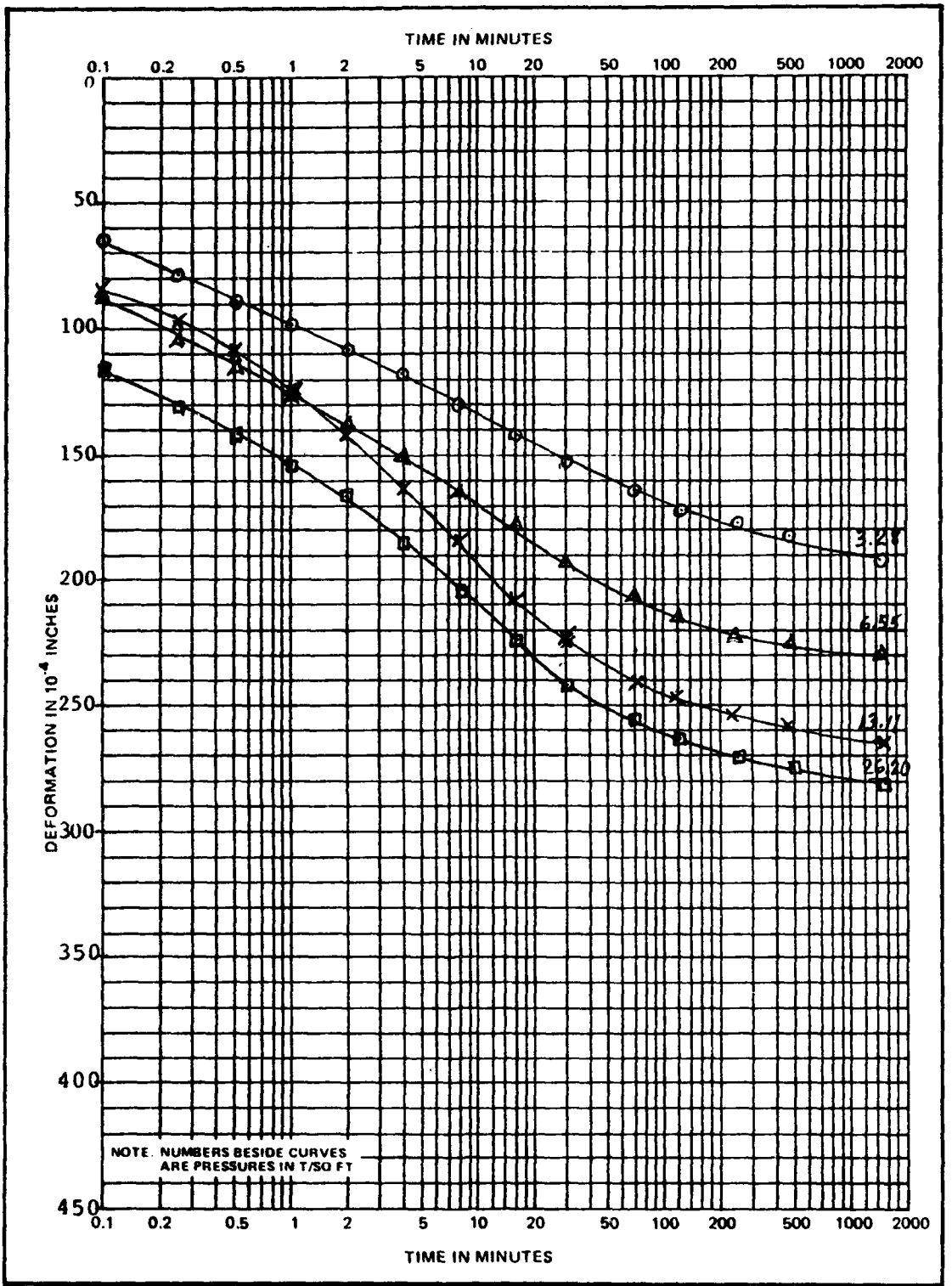
PLATE 66, PHASE III



PROJECT	SEVEN OAKS DAM - Impervious Core			
AREA	Combine 100045 & 100046 (Blend 3B) Phase III			
BORING NO.	CBT-86-34	SAMPLE NO 17 & 18	DEPTH EL 0.75'-5'	DATE July 1987

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST–TIME CURVES** (TRANSLUCENT)

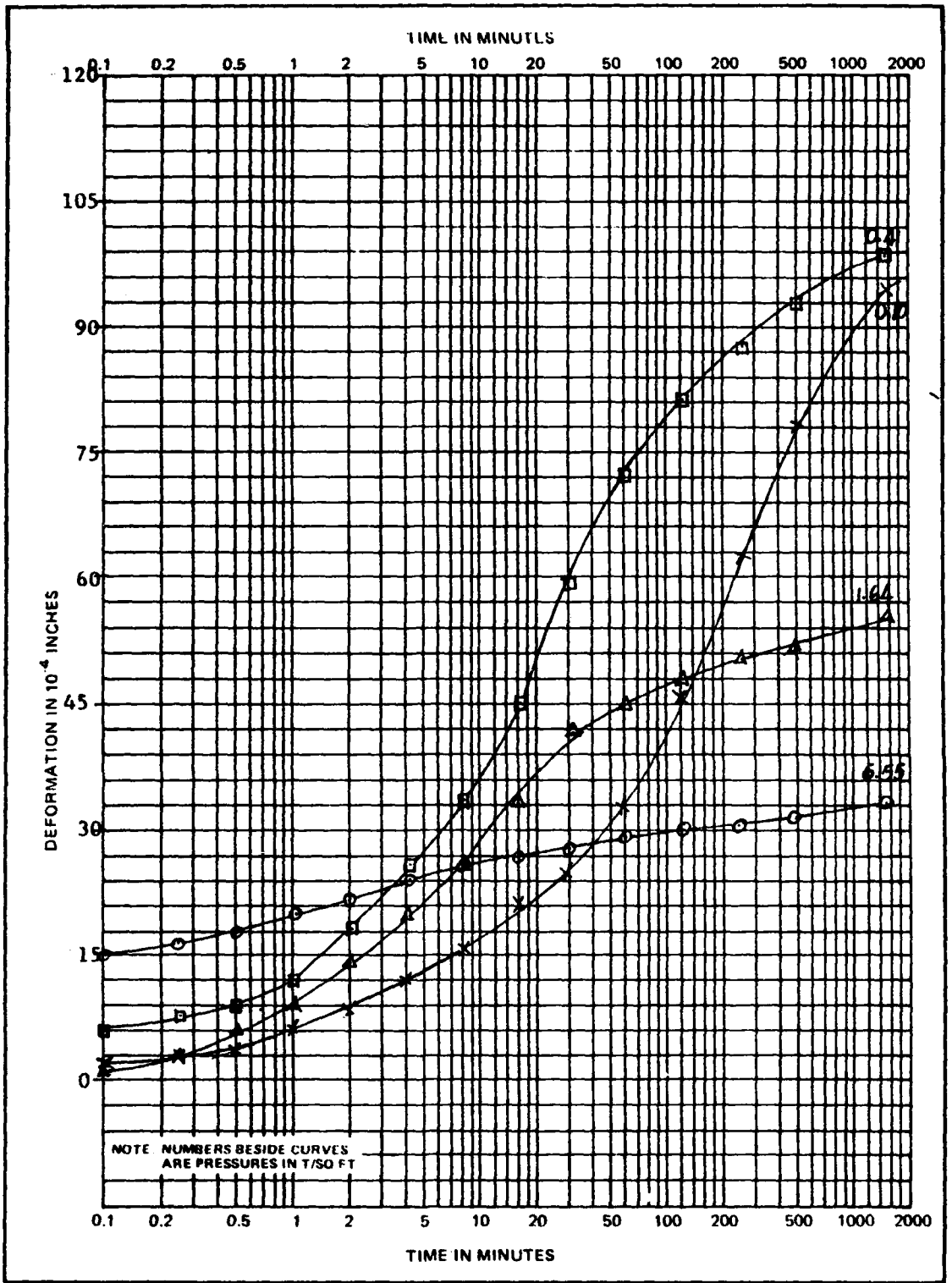
PLATE 67 , PHASE III



PROJECT SEVEN OAKS DAM - Imperious Core			
AREA Combine 100045 & 100046 (Blend 3B) Phase III			
BORING NO. CBT-86-34	SAMPLE NO 17 & 18	DEPTH EL 0.75'-5'	DATE July 1987

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST—TIME CURVES** (TRANSLUCENT)

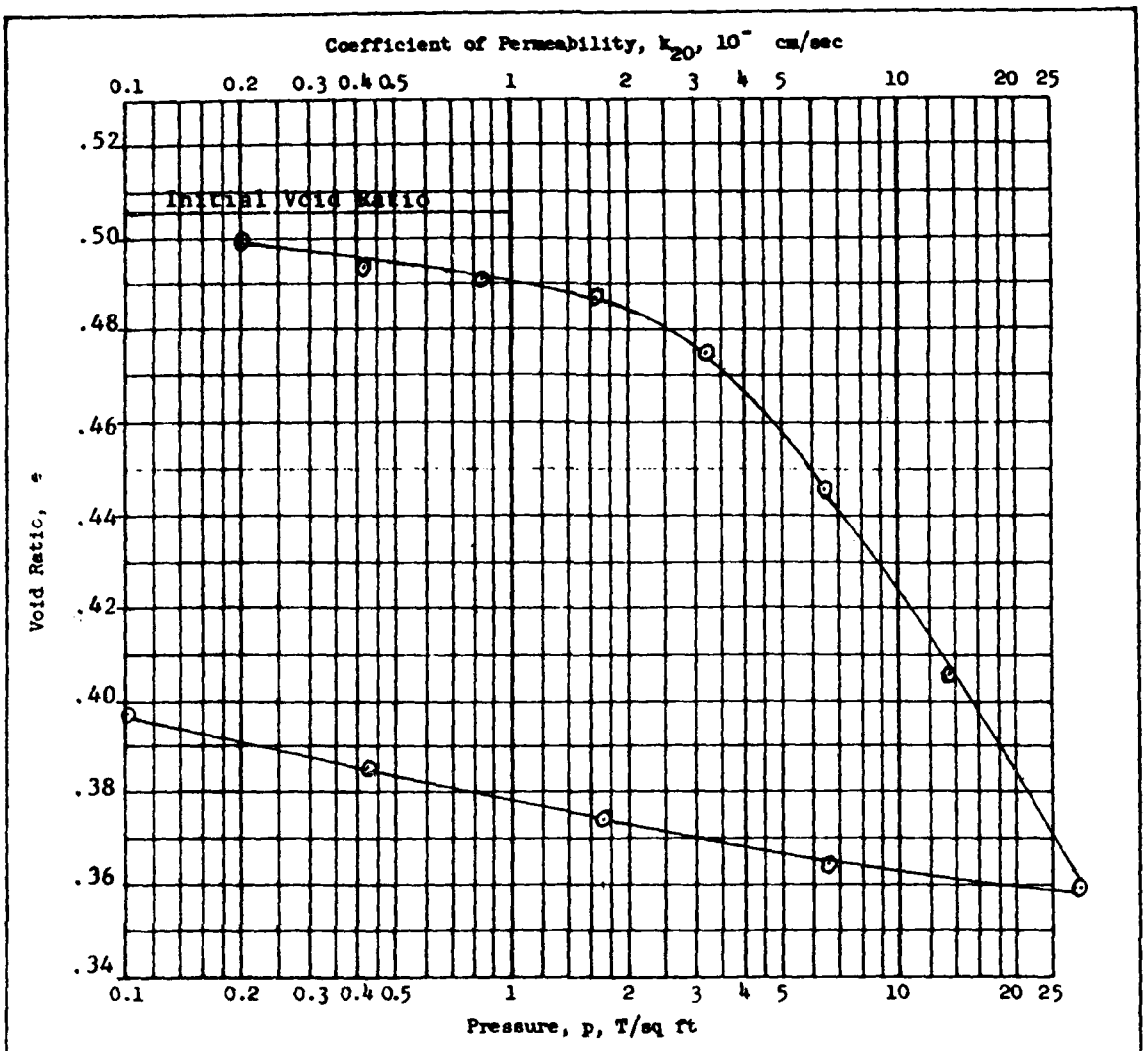
PLATE 68 ,PHASE III



PROJECT	SEVEN OAKS DAM - Impervious Core			
AREA	Combine 100045 & 100046 (Blend 3B) Phase III			
BORING NO.	CBT-86-34	SAMPLE NO 17 & 18	DEPTH EL 0.75'-5'	DATE July 1987

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST--TIME CURVES** (TRANSLUCENT)

PLATE 69, PHASE III



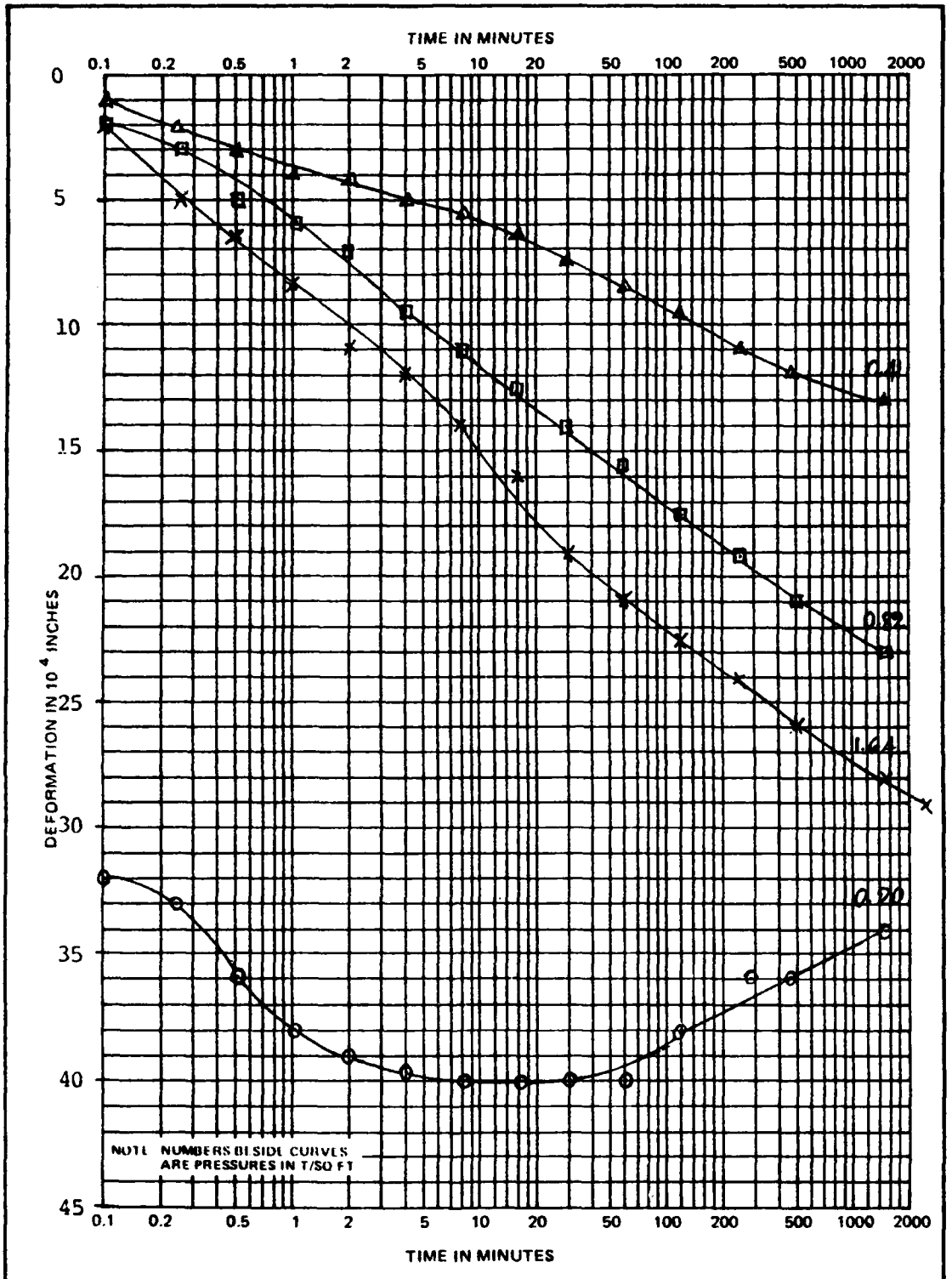
Type of Specimen Remolded		Before Test		After Test	
Diam 4.4 in.	Ht 1 in.	Water Content, w_o	12.3 %	w_f	13.9 %
Overburden Pressure, p_o T/sq ft		Void Ratio, e_o	0.505	e_f	0.417
Preconsol. Pressure, p_c T/sq ft		Saturation, S_o	65.1 %	S_f	89.5 %
Compression Index, C_c		Dry Density, γ_d	111.1 lb/ft^3		
Classification Sandy Clay (CL)		k_{20} at e_o =	x 10^{-} cm/sec		
LL 30	G_s 2.78	Project SEVEN OAKS DAM - Impervious Core (PHASE III)			
PL 17	D_{10}	Combine 100045 & 100046			
Remarks		Area (Blend 3B)			
Remolded to 98% max. density		Boring No. CBT-86-34		Sample No. 17 & 18	
@ opt. water content-2%		Depth El 0.75'-5'		Date July 1987	

CONSOLIDATION TEST REPORT

ENG FORM 2090 1 MAY 63 PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT)

GPO 936-736

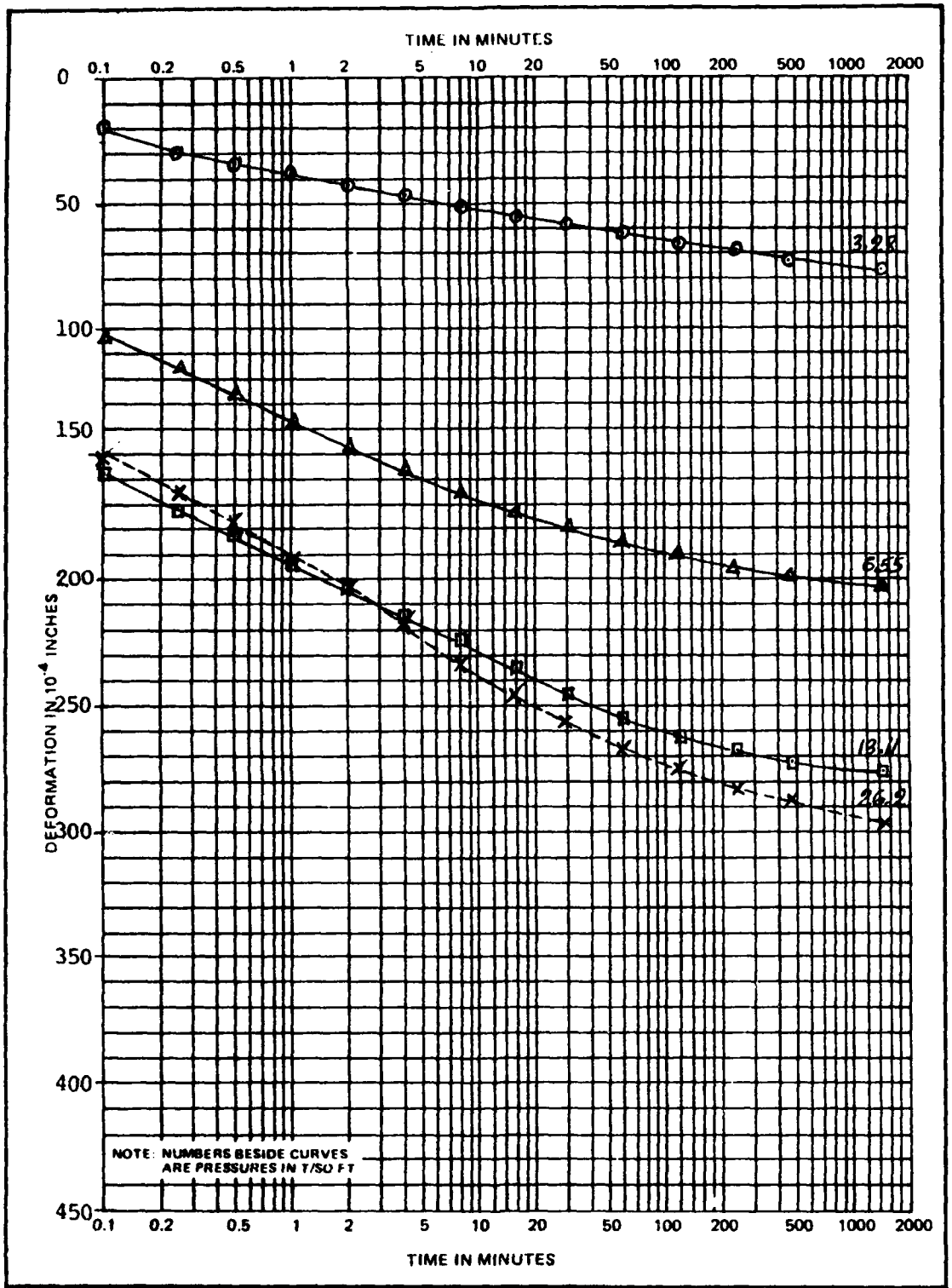
PLATE 70, PHASE III



PROJECT	SEVEN OAKS DAM - IMPERVIOUS CORE (PHASE III)		
AREA	Combine 100045 & 100046 (Blend 3B)		
BORING NO CBT-86-34	SAMPLE NO 17 & 18	DEPTH EL 0.75'-5'	DATE Jul 1987

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST—TIME CURVES** (TRANSLUCENT)

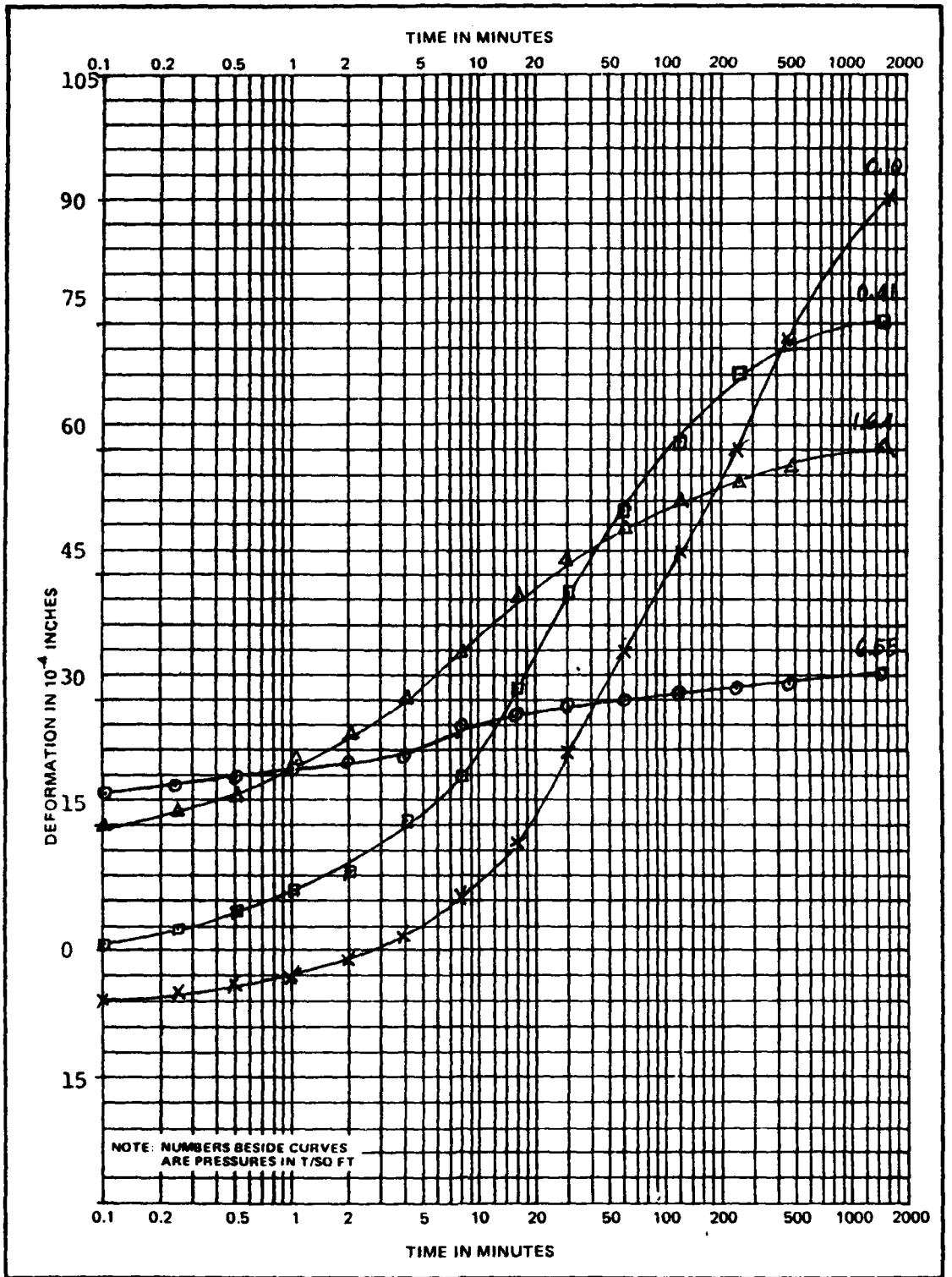
PLATE 71 , PHASE III



PROJECT	SEVEN OAKS DAM - IMPERVIOUS CORE (PHASE III) Combine 100045 & 100046 (Blend 3B)		
AREA			
BORING NO CBT-86-34	SAMPLE NO 17 & 18	DEPTH EL 0.75'-5'	DATE July 1987

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST--TIME CURVES** (TRANSLUCENT)

PLATE 72 , PHASE III



PROJECT	SEVEN OAKS DAM - IMPERVIOUS CORE (PHASE III)			
AREA	Combine 100045 & 100046 (Blend 3B)			
BORING NO. CBT-86-34	SAMPLE NO 17 & 18	DEPTH EL 0.75' - 5'	DATE July 1987	
ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE	**CONSOLIDATION TEST–TIME CURVES** (TRANSLUCENT)			

PLATE 73 , PHASE III

U.S ARMY ENGINEER DIVISION LABORATORY - - SOUTH PACIFIC DIVISION

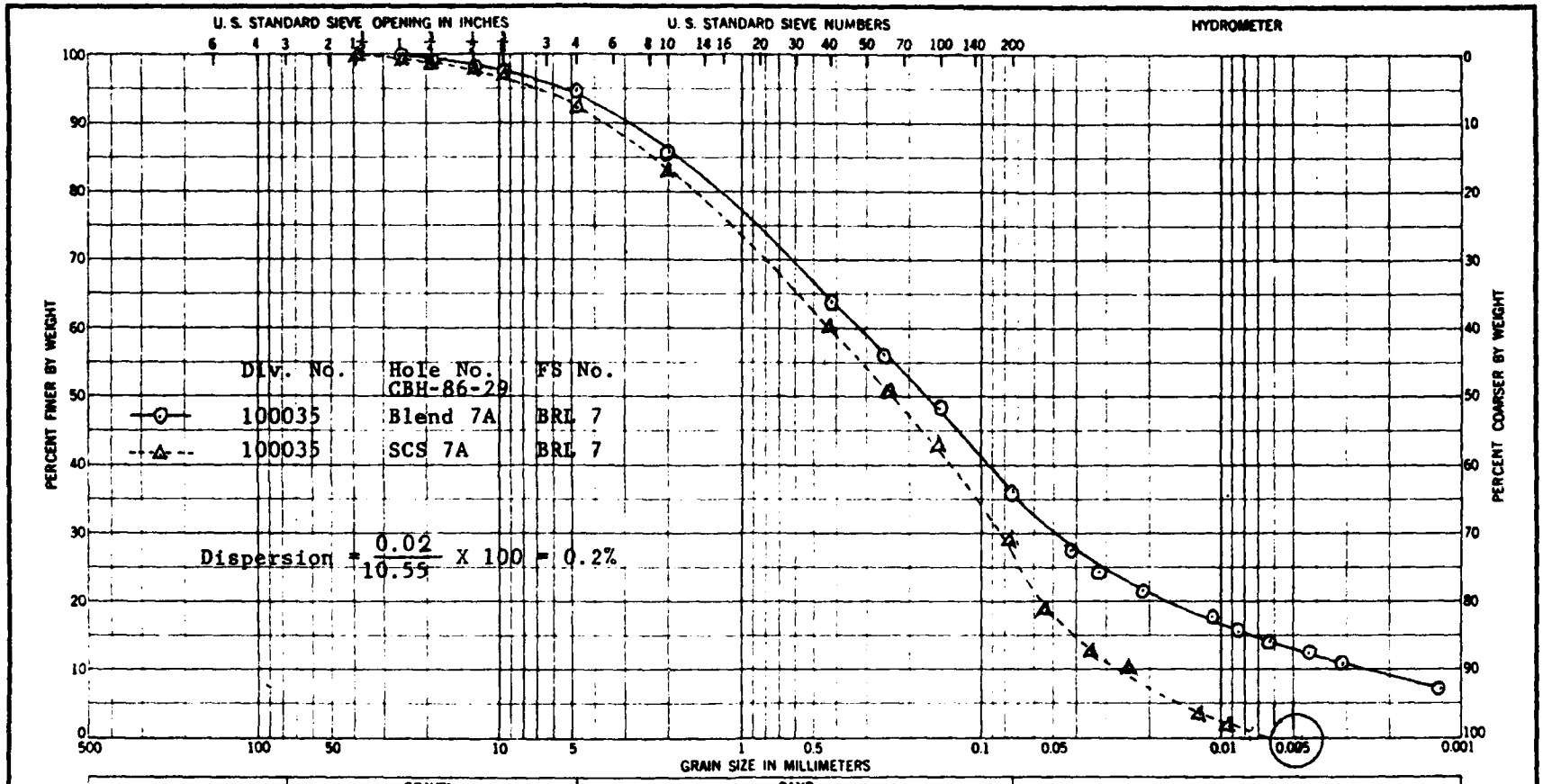
SOIL TEST RESULT SUMMARY

PROJECT: Upper Santa Ana River - Seven Oaks Dam (Mill Creek Impervious Borrow, Program 2: Phase IV) DATE: SEPTEMBER 1987

Division Number	Hole Number	Field Sample No.	Depth or Elevation From	Depth or Elevation To	Laboratory Descriptive Classification	Mechanical Analysis - % Finer: Gravel 1.5	Gravel 1	Gravel 3/4	Gravel 1/2	Gravel 3/8	Sand #4	Sand #10	Sand #40	Sand #60	Sand #100	Fines #200	Liquid Limit	Plasticity Index	Field Moist. %
100035	CBH-86-29 Blend 7A	BRL 7	0.0	10.0	Clayey Sand (SC-SM)	Sp.Gr. = 2.73	100	99	98	98	95	86	63	56	48	36	22	6	
100035	CBH-86-29 SCS 7A	BRL 7	0.0	10.0	Clayey Sand (SC-SM)		100	99	98	98	85	84	61	52	44	30	22	6	

SPD FORM 66A

PLATE 74, PHASE IV



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
BRL7	0.0-10.0	Clayey Sand (SC-SM)		22	16	6

GRADATION CURVES

Project UPPER SANTA ANA RIVER
Seven Oaks Dam, Program 2
Area Impervious Core (Phase IV)
Boring No. Mill Creek Impervious Borrow
Date September 1987

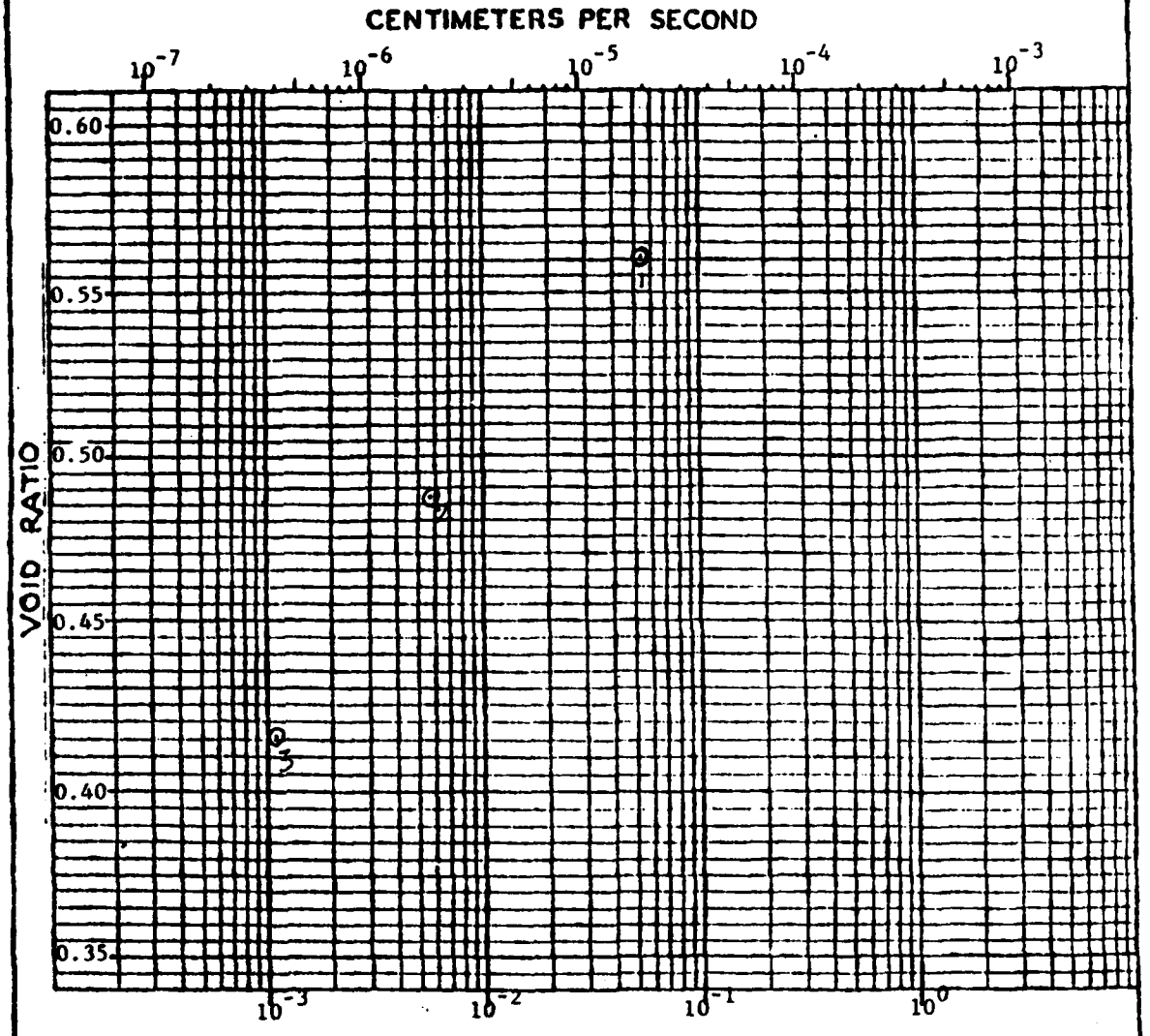
PLATE 75, PHASE IV

ENG FORM 1 MAY 63 2087

CORPS OF ENGINEERS — U. S. ARMY

SOUTH PACIFIC DIVISION LABORATORY

PERMEABILITY



REMARKS: C_1=90% C_2=95% C_3=100% Max. density, Optimum Moisture

Points	SPECIMEN DIAM. IN.	HT. IN.	MAX. PARTICLE	CONDITION
1	6.05	5.96	2 in.	Undisturbed
2	5.95	5.87		
3	5.86	5.78		

DISTRICT: LOS ANGELES

PROJECT: SEVEN OAKS DAM-IMPERVIOUS CORE Program 2 (Phase IV)

Points	DIV. NO.	HOLE NO.	F.S. NO.	DEPTH FROM	DEPTH TO
1	100030	CBT86-30	Bar	5.0'	11.0'
2	Blend		2		
3	2A				

TESTED	COMPUTED	DRAWN	CHECKED
FN	FN	CRK	

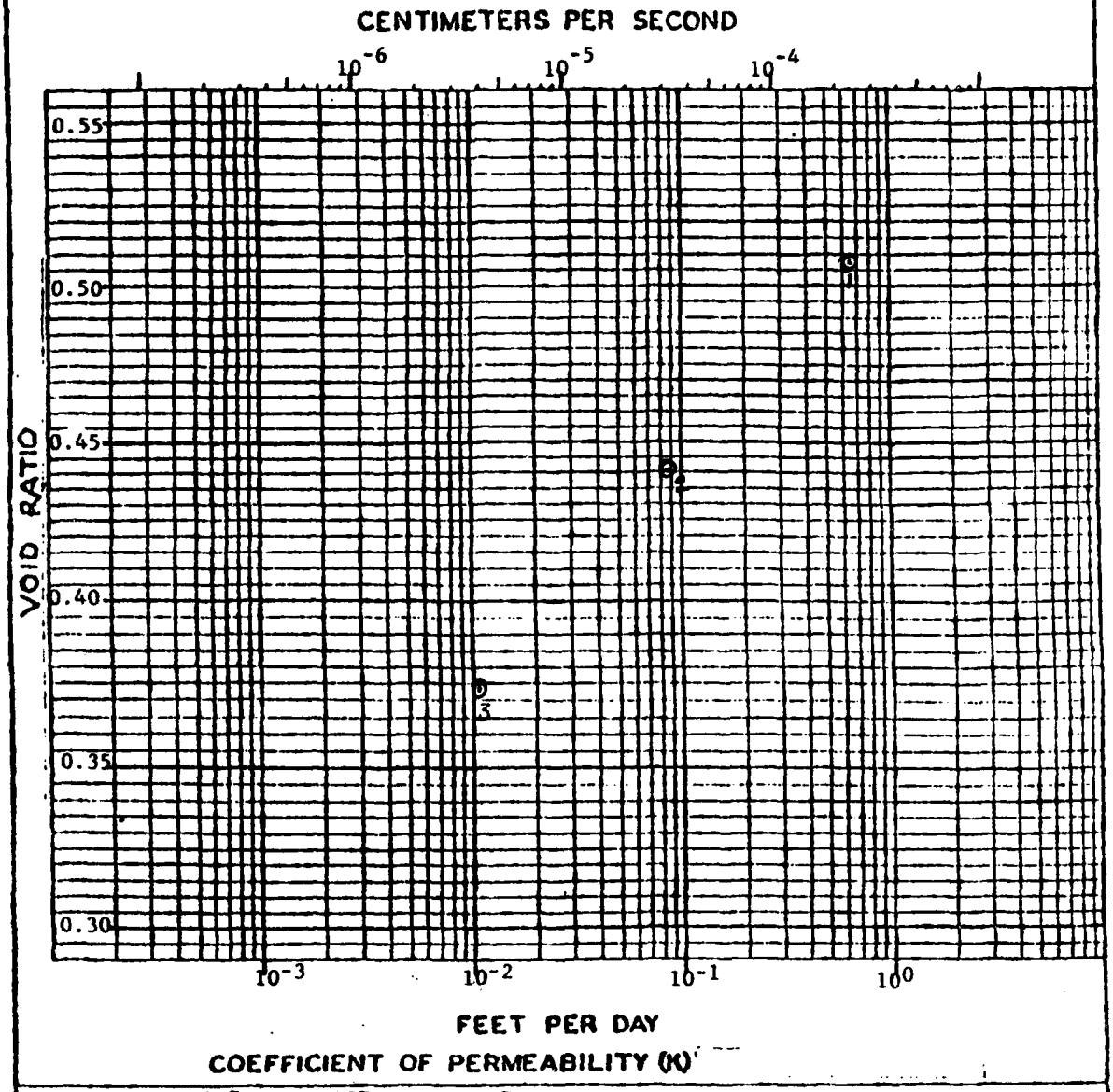
SPD FORM 4 OCT 55 67 (CIVIL) REPLACES PSD 67 (CIVIL), 20 JULY 49, WHICH MAY BE USED.

PLATE 76, PHASE IV

CORPS OF ENGINEERS — U. S. ARMY

SOUTH PACIFIC DIVISION LABORATORY

PERMEABILITY



REMARKS: C_1=90% C_2=95% C_3=100% Max density, optimum moisture

Points	SPECIMEN DIAM. IN.	HT. IN.	MAX. PARTICLE	CONDITION
1	6.00	6.03	1 in.	Undisturbed
2	5.90	5.94		
3	5.81	5.86		

DISTRICT: LOS ANGELES

PROJECT: SEVEN OAKS DAM-IMPERVIOUS CORE Program 2 (Phase IV)

Points	DIV. NO.	HOLE NO.	F.S. NO.	DEPTH FROM	DEPTH TO
1	100035	CBH86-29	Bar	0.0'	10.0'
2	Blend		7		
3	7A				

TESTED	COMPUTED	DRAWN	CHECKED
FN	FN	CRK	

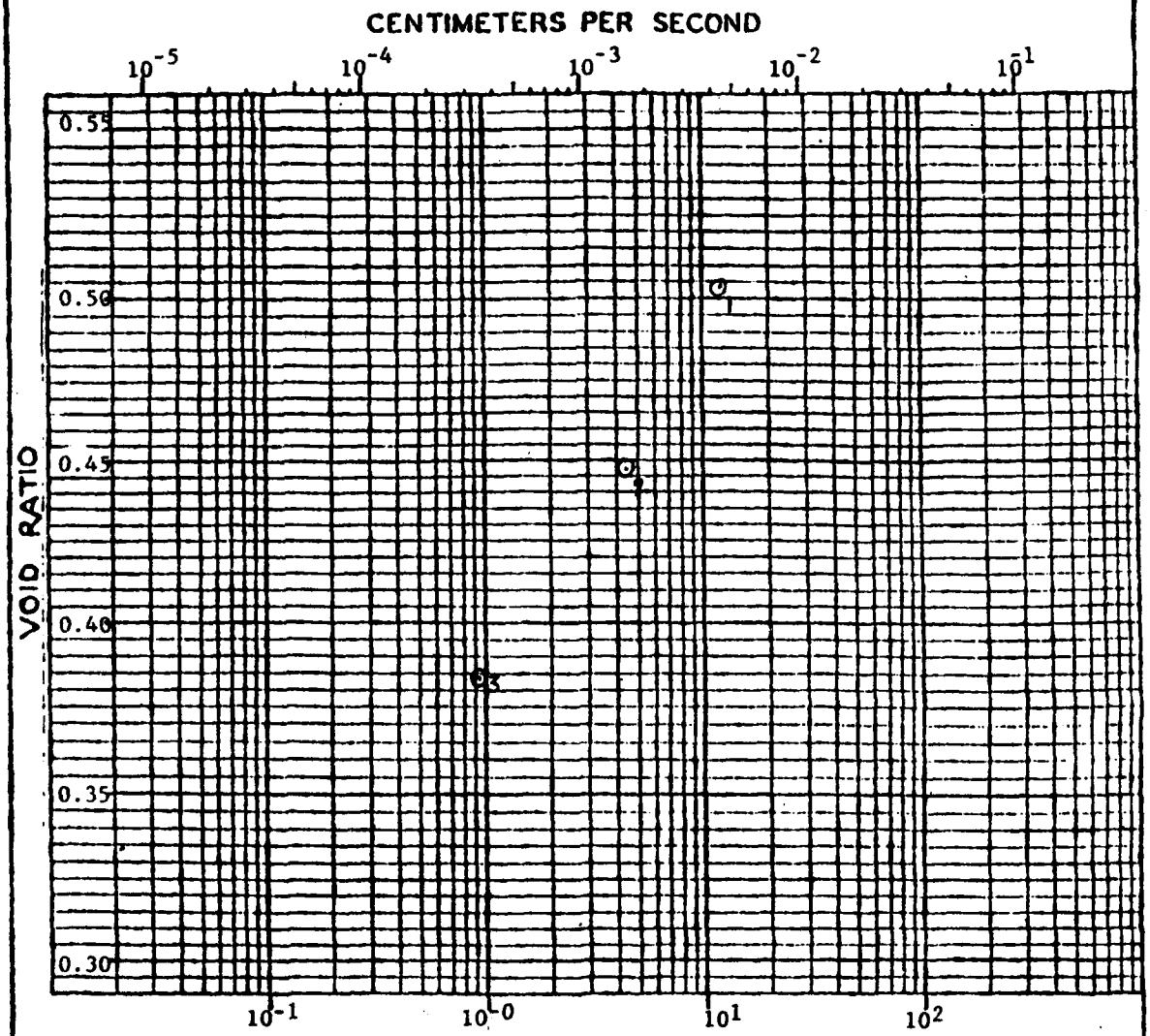
SPD FORM 67 (CIVIL) 4 OCT 55 REPLACES PSD 67 (CIVIL), 20 JULY 49, WHICH MAY BE USED.

PLATE 77, PHASED

CORPS OF ENGINEERS — U. S. ARMY

SOUTH PACIFIC DIVISION LABORATORY

PERMEABILITY



REMARKS: e_1=90% ; e_2=95% ; e_3=100% ; Max. density, optimum moisture

Points	SPECIMEN DIAM. IN.	HT. IN.	MAX. PARTICLE	CONDITION
1	6.00	6.00	6 in.	Undisturbed
2	5.93	5.89		
3	5.84	5.81		

DISTRICT: LOS ANGELES

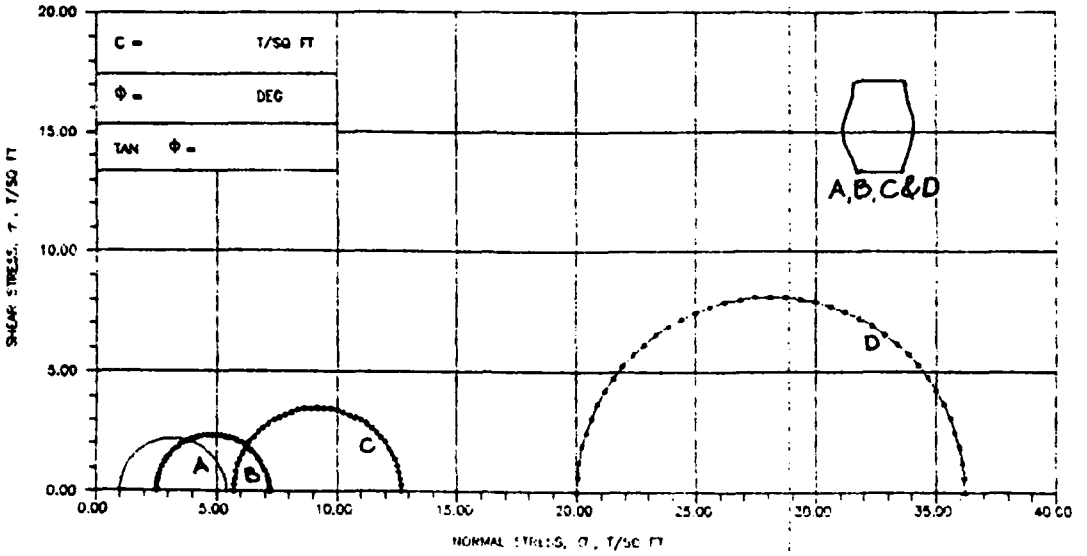
PROJECT: SEVEN OAKS DAM - IMPERVIOUS CORE Program 2 (Phase IV)

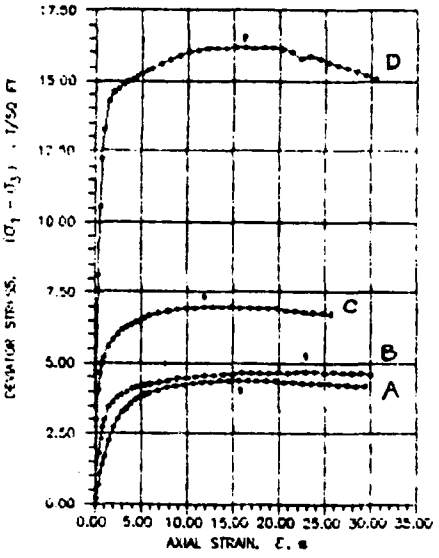
Points	DIV. NO.	HOLE NO.	F.S. NO.	DEPTH FROM	DEPTH TO
1	100050	CBH86-38	Bar	7.0	12.0
2	Blend		22		
3	6A				

TESTED	COMPUTED	DRAWN	CHECKED
FN	FN	CRK	

SPD Form 4 Oct 55 67 (CIVIL) REPLACES PSD 67 (CIVIL), 20 JULY 49, WHICH MAY BE USED.

PLATE 78 PHASE IV





SPECIMEN NO.			A	B	C	D
INITIAL	WATER CONTENT, %	w_o	10.2	11.4	10.5	10.3
	DRY DENSITY LB/CU FT	γ_{d_o}	117.0	116.9	116.7	117.0
	SATURATION, %	S_o	64	71	65	64
	VOID RATIO	e_o	0.430	0.430	0.433	0.430
BEFORE SHEAR	WATER CONTENT, %	w_c	15.8	15.2	15.1	12.7
	DRY DENSITY LB/CU FT	γ_{d_c}	117.5	118.9	119.0	124.7
	SATURATION, %	S_c	100	100	100	100
	VOID RATIO	e_c	0.423	0.407	0.405	0.342
	FINAL BACK PRESSURE, T/SQ FT	U_b	5.8	5.8	5.8	5.8
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	1.0	2.5	5.7	20.0
MAXIMUM DEVIATOR STRESS, T/SQ FT	$(\sigma_1 - \sigma_3)_{MAX}$		4.39	4.61	6.98	16.17
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	321	312	328	214
ULTIMATE DEVIATOR STRESS, T/SQ FT	$(\sigma_1 - \sigma_3)_{ULT}$		–	–	–	–
INITIAL DIAMETER, IN.		D_o	2.80	2.80	2.80	2.80
INITIAL HEIGHT, IN.		H_o	6.45	6.45	6.45	6.45

CONTROLLED STRAIN TEST: Yes

DESCRIPTION OF SPECIMEN: Clayey Sand(SC)

LL	PL	PI	G_s	TYPE OF SPECIMEN	TYPE OF TEST
33	20	13	2.68	Remolded	"R"

REMARKS: Div.No. 100030
Remolded 98% max. density @ 2% below Opt water.
Max density = 118.9 pcf
Opt. water = 12.7%

PROJECT: SEVEN OAKS DAM
Mill Creek Impervious Borrow
BORING NO. CBT-86-30 SAMPLE NO Blend 2A
(Program 2,Phase IV)
LABORATORY SPD Lab DATE September 1987

TRIAXIAL COMPRESSION TEST REPORT

ENG FORM NO. 2089
REV. JUNE 1970

PLATE 79, PHASE IV

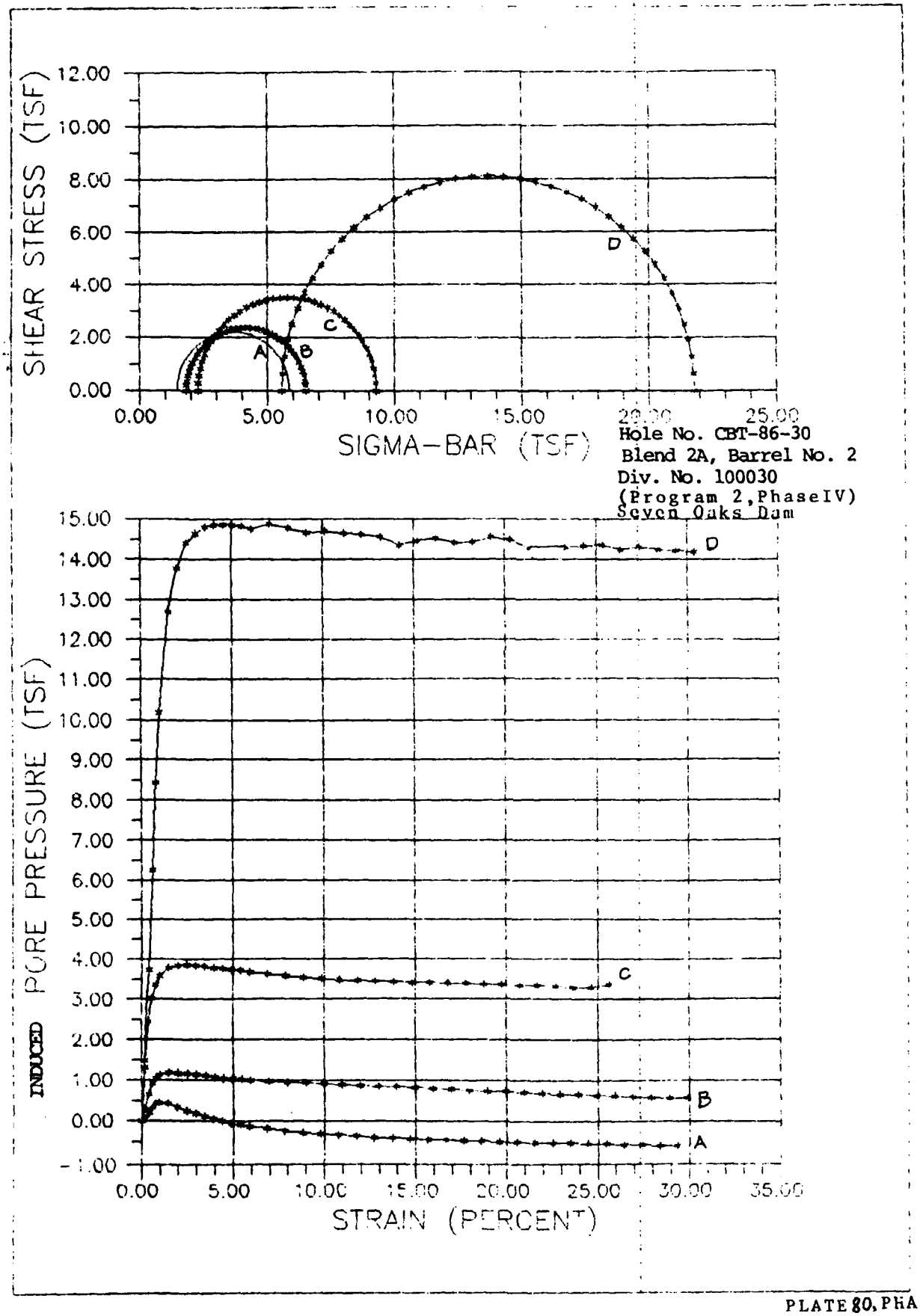
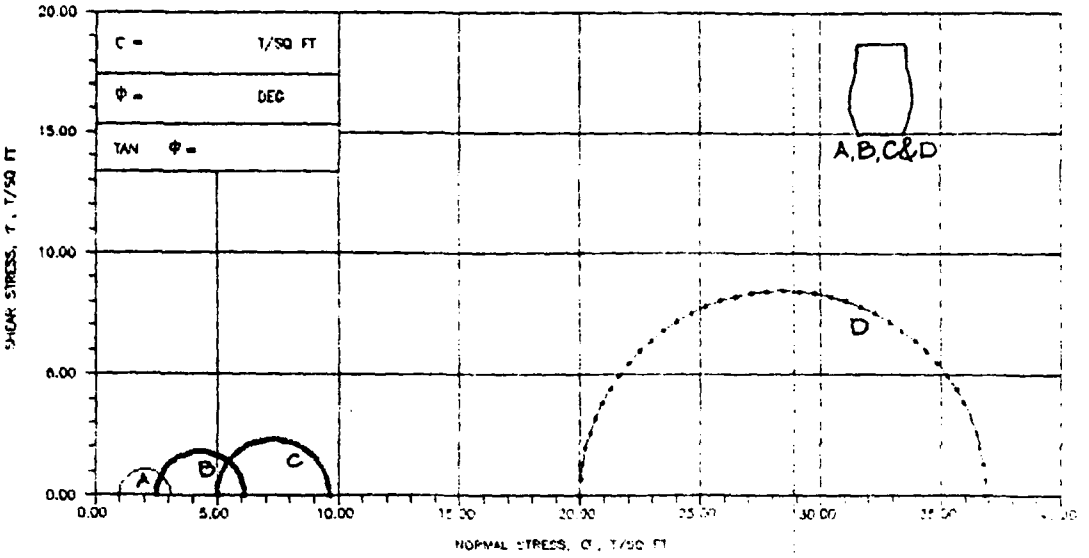
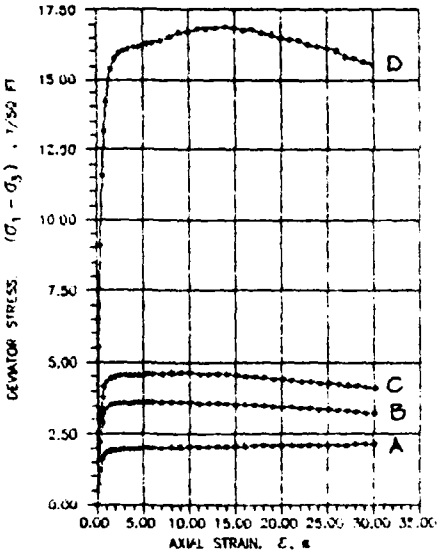


PLATE 80, PHASE IV





SPECIMEN NO.			A	B	C	D
INITIAL	WATER CONTENT, %	w_o	9.5	9.5	9.4	9.5
	DRY DENSITY LB/CU FT	γ_{d_o}	119.8	119.8	119.9	119.8
	SATURATION, %	S_o	62	62	61	61
	VOID RATIO	e_o	0.422	0.423	0.421	0.422
BEFORE SHEAR	WATER CONTENT, %	w_c	15.0	14.6	14.1	12.5
	DRY DENSITY LB/CU FT	γ_d	120.8	121.8	122.9	127.1
	SATURATION, %	S_c	100	100	100	100
	VOID RATIO	e_c	0.411	0.399	0.386	0.341
	FINAL BACK PRESSURE, T/SQ FT	u_b	5.8	5.8	5.8	5.8
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	1.0	2.5	5.0	20.0
MAXIMUM DEVIATOR STRESS, T/SQ FT	$(\sigma_1 - \sigma_3)_{MAX}$		2.10	3.54	4.63	16.87
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$ MIN		t_f	218	178	120	190
ULTIMATE DEVIATOR STRESS, T/SQ FT	$(\sigma_1 - \sigma_3)_{ULT}$		-	-	-	-
INITIAL DIAMETER, IN		D_o	2.80	2.80	2.80	2.80
INITIAL HEIGHT, IN.		H_o	6.45	6.45	6.45	6.45

CONTROLLED STRAIN TEST: Yes

DESCRIPTION OF SPECIMEN: Clayey Sand (SC)

LL 23	PL 16	PI 7	G_s 2.73	TYPE OF SPECIMEN Remolded	TYPE OF TEST "R"

REMARKS: Remolded to 98% maximum density @ 2% below optimum water. Max. density=121.9pcf. Opt. water=11.7%. Div. No. 100035, Blend 7A

PROJECT: SEVEN OAKS DAM
Mill Creek Impervious Borrow

BORING NO CBH 86-29 | SAMPLE NO Barrel No. 7

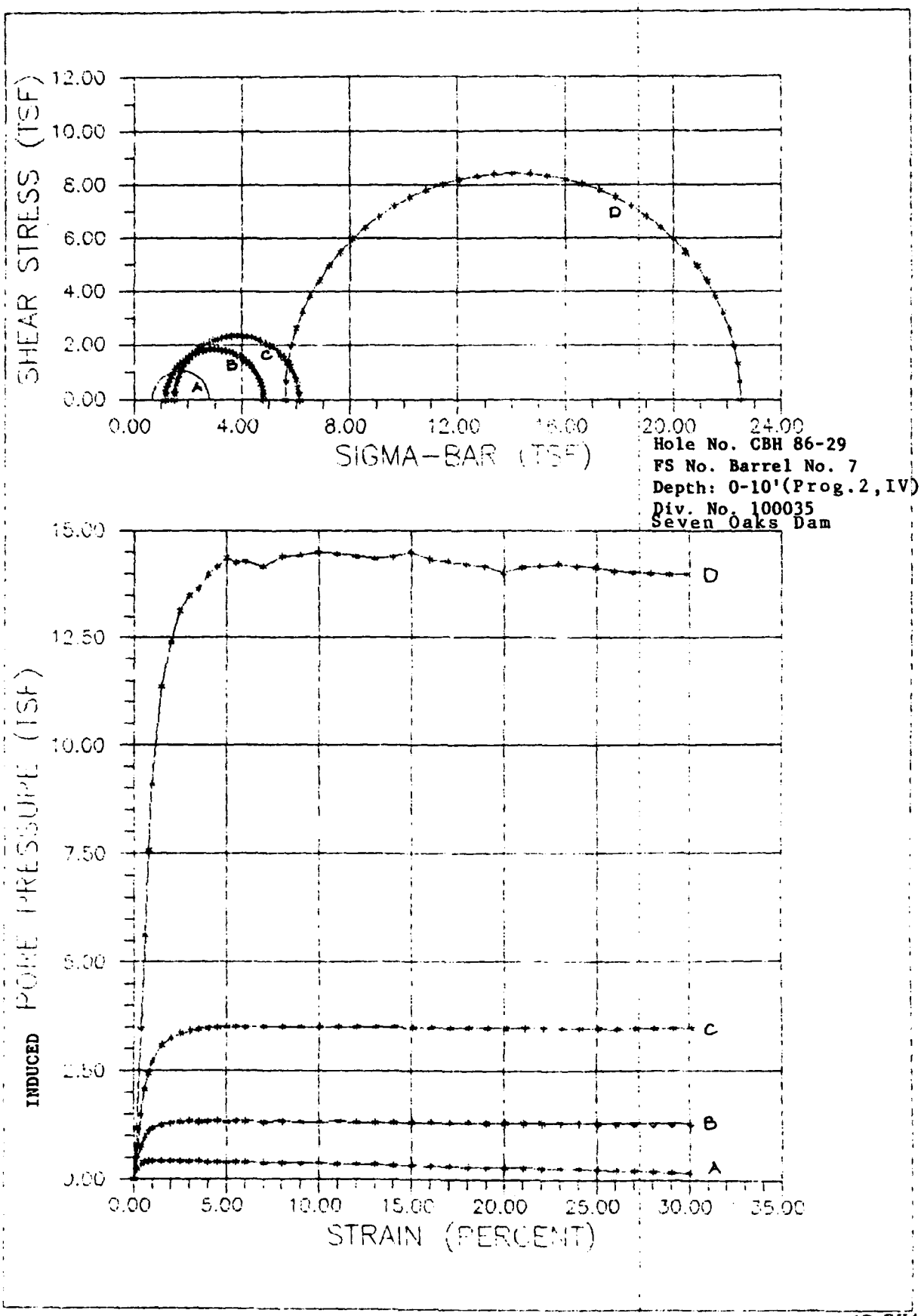
Depth: 0-10' (Program 2, Phase IV)

LABORATORY SPD | DATE Sept. 1987

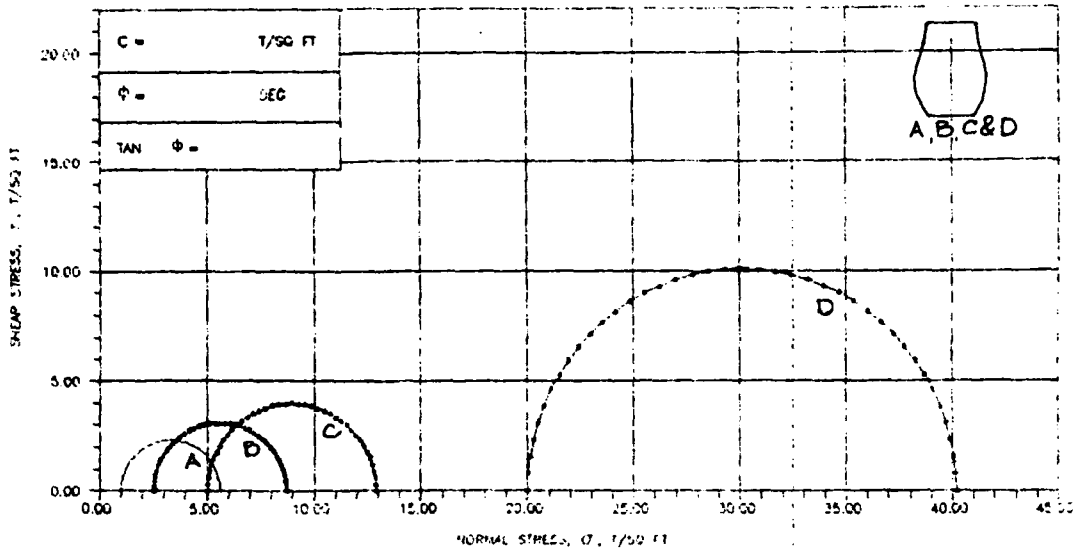
TRIAXIAL COMPRESSION TEST REPORT

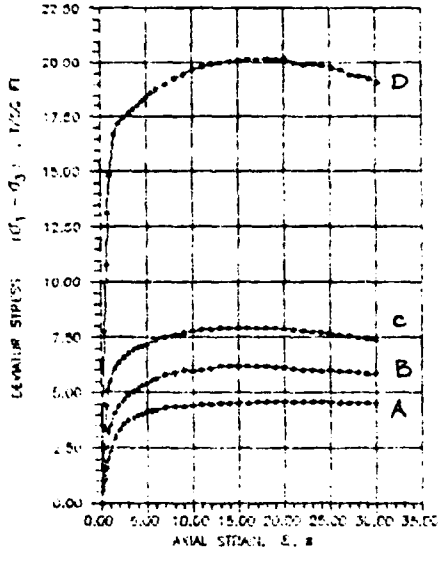
ENG FORM NO. 2089
REV JUNE 1970

PLATE 81, PHASE I



Hole No. CBH 86-29
FS No. Barrel No. 7
Depth: 0-10'(Prog.2,IV)
Div. No. 100035
Seven Oaks Dam





SPECIMEN NO.		A	B	C	D
INITIAL — WATER CONTENT, %	w_o	9.0	8.9	8.8	8.9
INITIAL — DRY DENSITY LB/CU FT	γ_{d_o}	118.8	118.9	119.0	118.9
INITIAL — SATURATION, %	S_o	59	59	58	59
INITIAL — VOID RATIO	e_o	0.408	0.407	0.406	0.407
BEFORE SHEAR — WATER CONTENT, %	w_c	14.8	14.3	14.0	12.0
BEFORE SHEAR — DRY DENSITY LB/CU FT	γ_{d_c}	119.7	120.9	121.6	126.5
BEFORE SHEAR — SATURATION, %	S_c	100	100	100	100
BEFORE SHEAR — VOID RATIO	e_c	0.397	0.383	0.375	0.322
FINAL BACK PRESSURE, T/SQ FT	u_c	5.8	5.8	5.8	5.8
MINOR PRINCIPAL STRESS, T/SQ FT	σ_3	1.0	2.5	5.0	20.0
MAXIMUM DEVIATOR STRESS, T/SQ FT	$(\sigma_1 - \sigma_3)_{MAX}$	4.55	6.20	7.91	20.14
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN	t_f	215	212	169	188
ULTIMATE DEVIATOR STRESS, T/SQ FT	$(\sigma_1 - \sigma_3)_{ULT}$	-	-	-	-
INITIAL DIAMETER, IN	D_o	2.80	2.80	2.80	2.80
INITIAL HEIGHT, IN.	H_o	6.45	6.45	6.45	6.45

CONSOLIDATED STRAIN TEST: Yes

DESCRIPTION OF SPECIMEN: Silty Sand (SM)

LL | PL | PI: NP | G_s: 2.68 | TYPE OF SPECIMEN: Remolded | TYPE OF TEST: "R"

REMARKS: Div. No. 100050
Remolded to 98% max. density
@ 2% below optimum water

Max. density = 121.5 pcf
Opt. water = 10.7%

PROJECT: SEVEN OAKS DAM
Mill Creek Impervious Borrow
BORING NO.: CBT-86-38 | SAMPLE NO.: Blend 6A
Depth: 7'-12' (Program 2, Phase IV)
LABORATORY: SPD | DATE: September 1987

TRIAXIAL COMPRESSION TEST REPORT

END FORM NO. 2089
REV. JUNE 1970

PLATE 83, PHASE IV

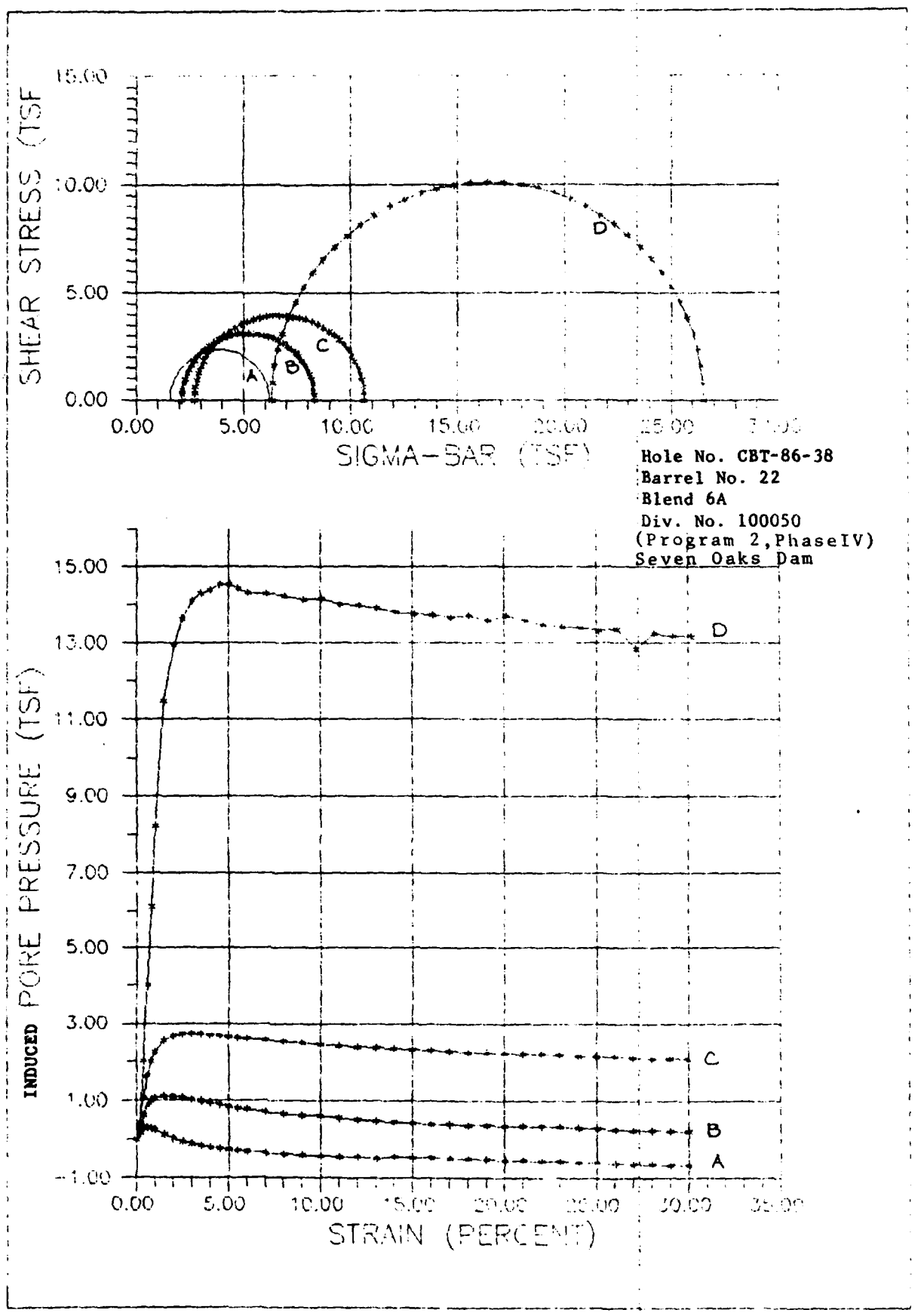
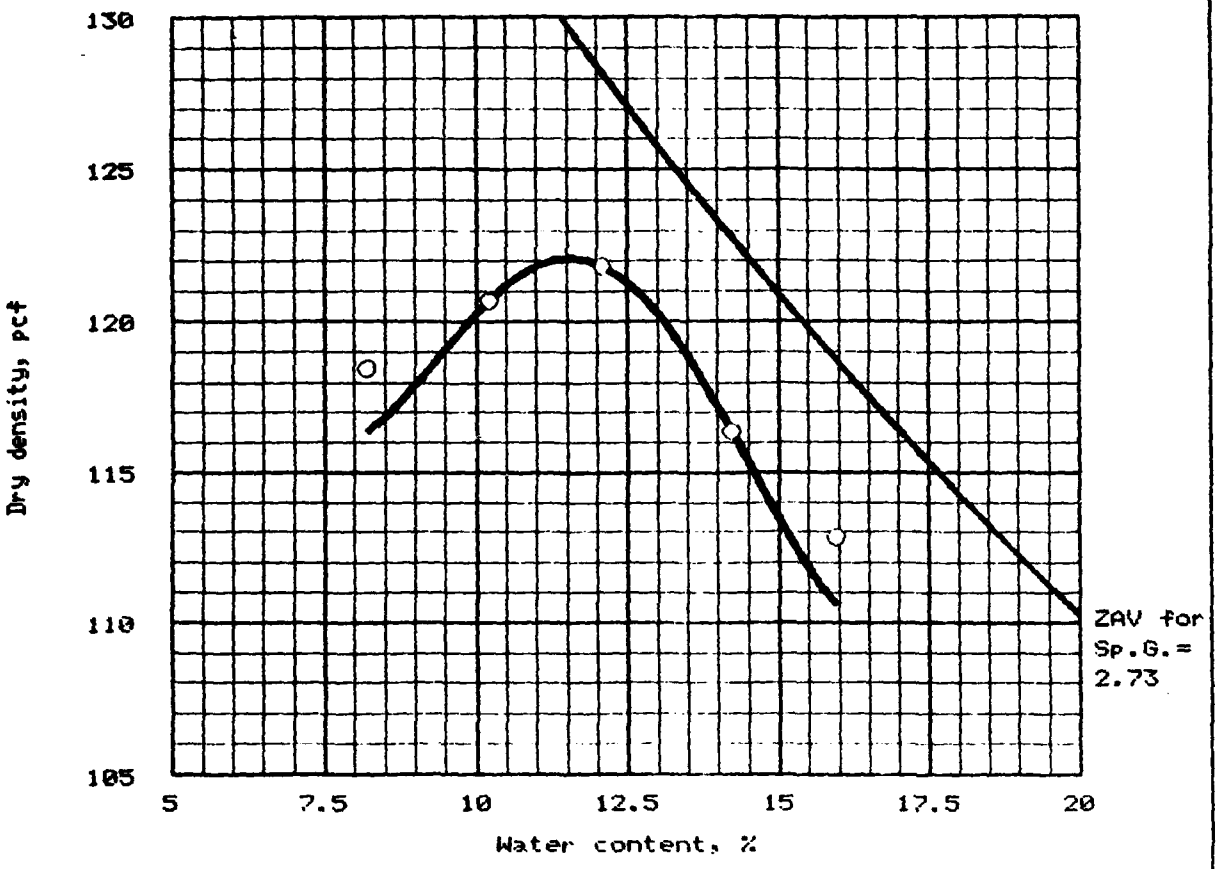


PLATE 84 PHASE IV

COMPACTION TEST REPORT



"Standard" Proctor, ASTM D 698-78, Method A

Elev/ Depth	USCS Classification	Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
0-10	Clayey Sand (SC-SM)		2.73	22	6	5 %	36 %

TEST RESULTS	MATERIAL DESCRIPTION
Optimum moisture = 11.6 % Maximum dry density = 122.1 pcf	Remolded

Division No.: 100035 F.S. No. Blend 7 A
Project: 7 Oaks Dam, Mill Creek Impervious Borrow
Location: Impervious Core, Program 2
Hole No. CBH-86-29
Date: 9-24-1987

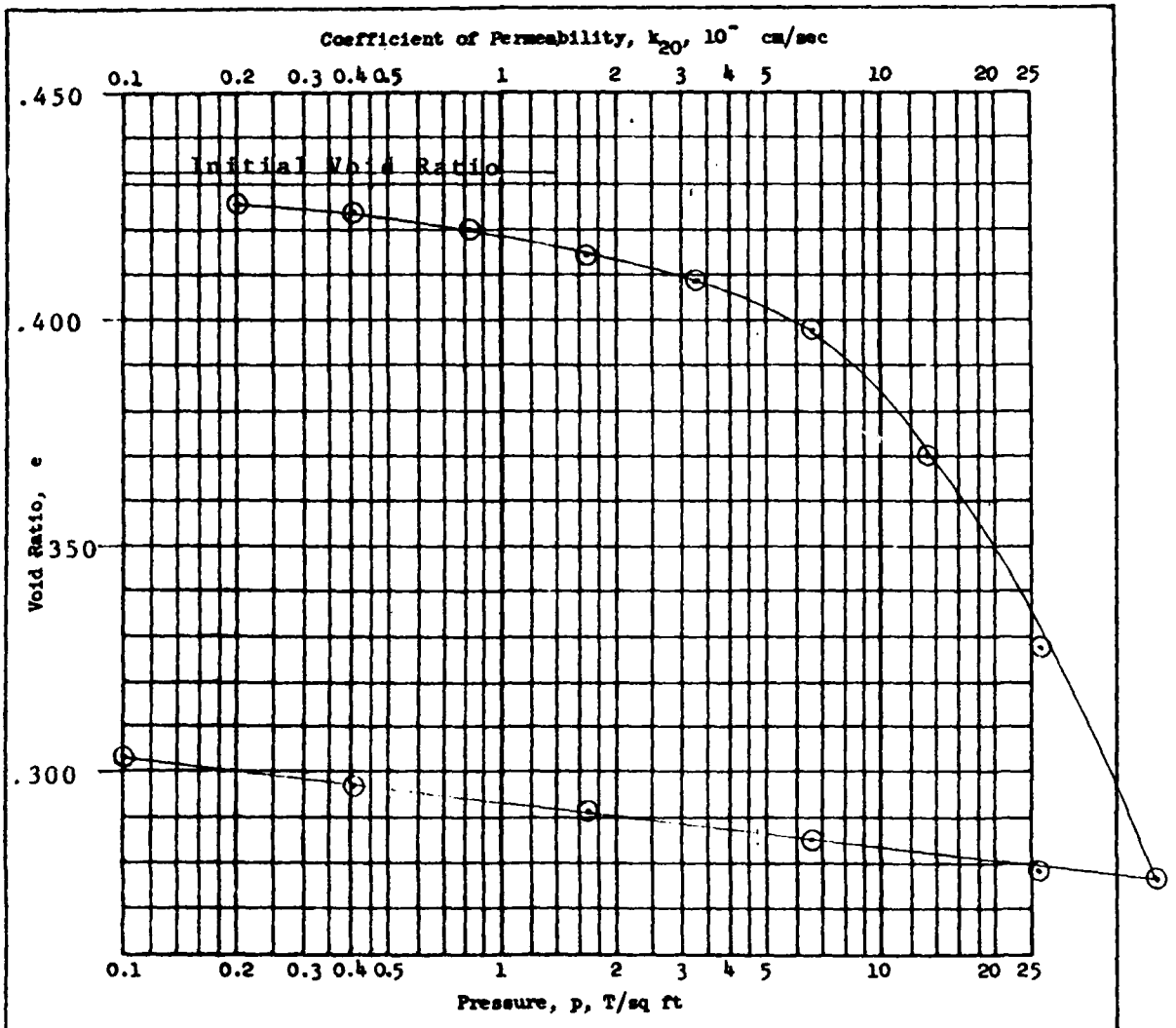
Remarks:

Barrel No. 7 (Phase IV)

COMPACTION TEST REPORT
CORPS OF ENGINEERS - SPD

Plate No. 90

PHASE IV

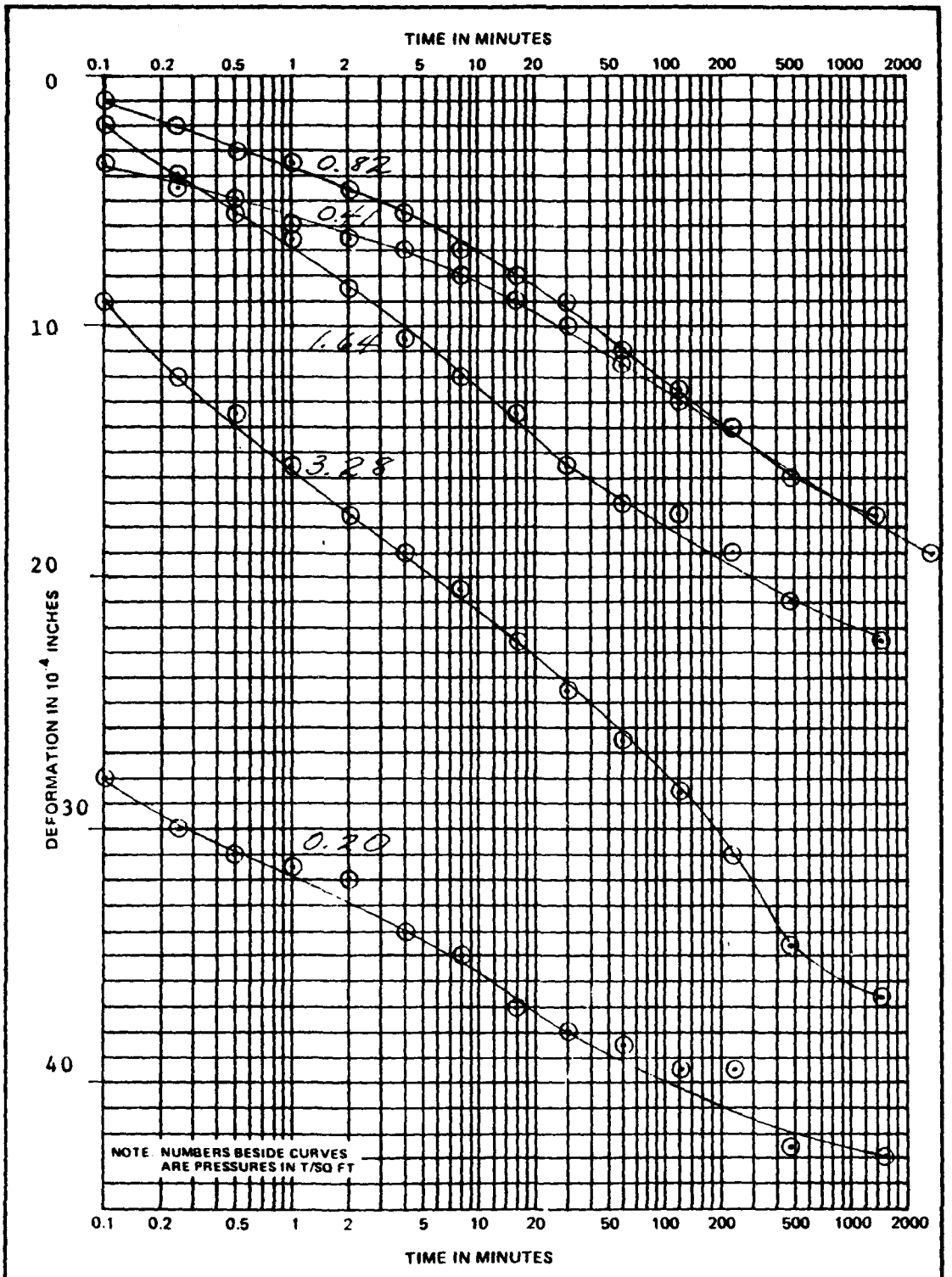


Type of Specimen Remolded		Before Test		After Test	
Diam 4.4 in.	Ht 1.0 in.	Water Content, w_o	12.5 %	w_f	11.7 %
Overburden Pressure, p_o T/sq ft		Void Ratio, e_o	.432	e_f	.315
Preconsol. Pressure, p_c T/sq ft		Saturation, S_o	77.6 %	S_f	99.2 %
Compression Index, C_c		Dry Density, γ_d	116.7 lb/ft^3		
Classification Clayey Sand(SC)		k_{20} at e_o =	x 10^{-} cm/sec		
LL 33	G_s 2.67	Project Seven Oaks Dam			
PL 20	D_{10}	Mill Creek Impervious Borrow			
Remarks Div. No. 100030		Area (Phase IV, Program 2)			
Remolded to 98% maximum		Boring No. CBT-86-30		Sample No. Blend 2A	
Density at Optimum		Depth El 5' - 11'		Date Sept., 1987	
		CONSOLIDATION TEST REPORT			

ENG FORM 1 MAY 63 2090 PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT)

PLATE 91 PHASE IV

GPO 936-736



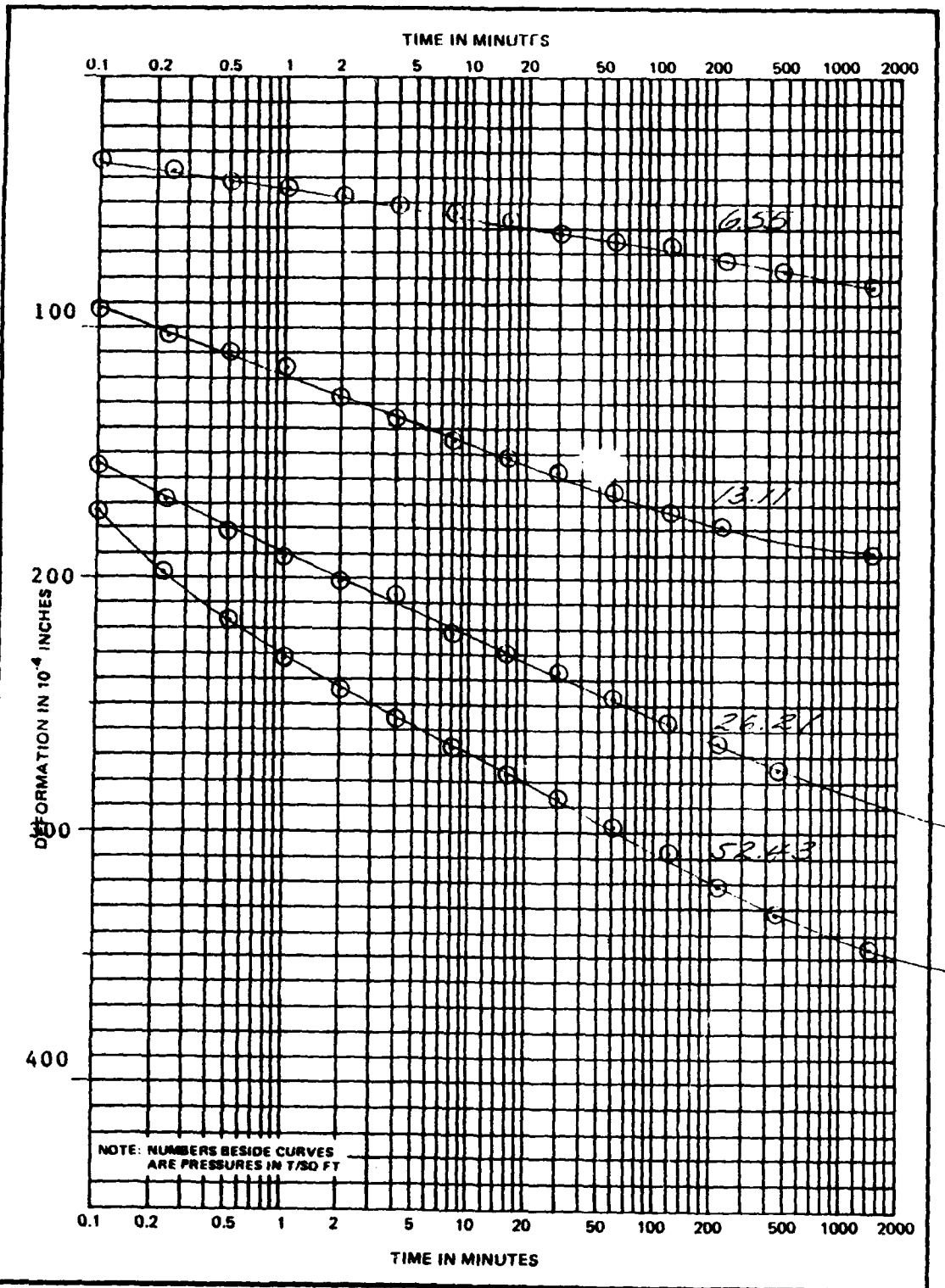
PROJECT Seven Oaks Dam, Div. No. 100030

AREA (Mill Creek-Impervious Borrow, Program 2; Phase IV)

BORING NO. CBT-86-30 | SAMPLE NO Blend 2 | DEPTH EL 5.0'-11.0' | DATE Sept 1987

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST–TIME CURVES** (TRANSLUCENT)

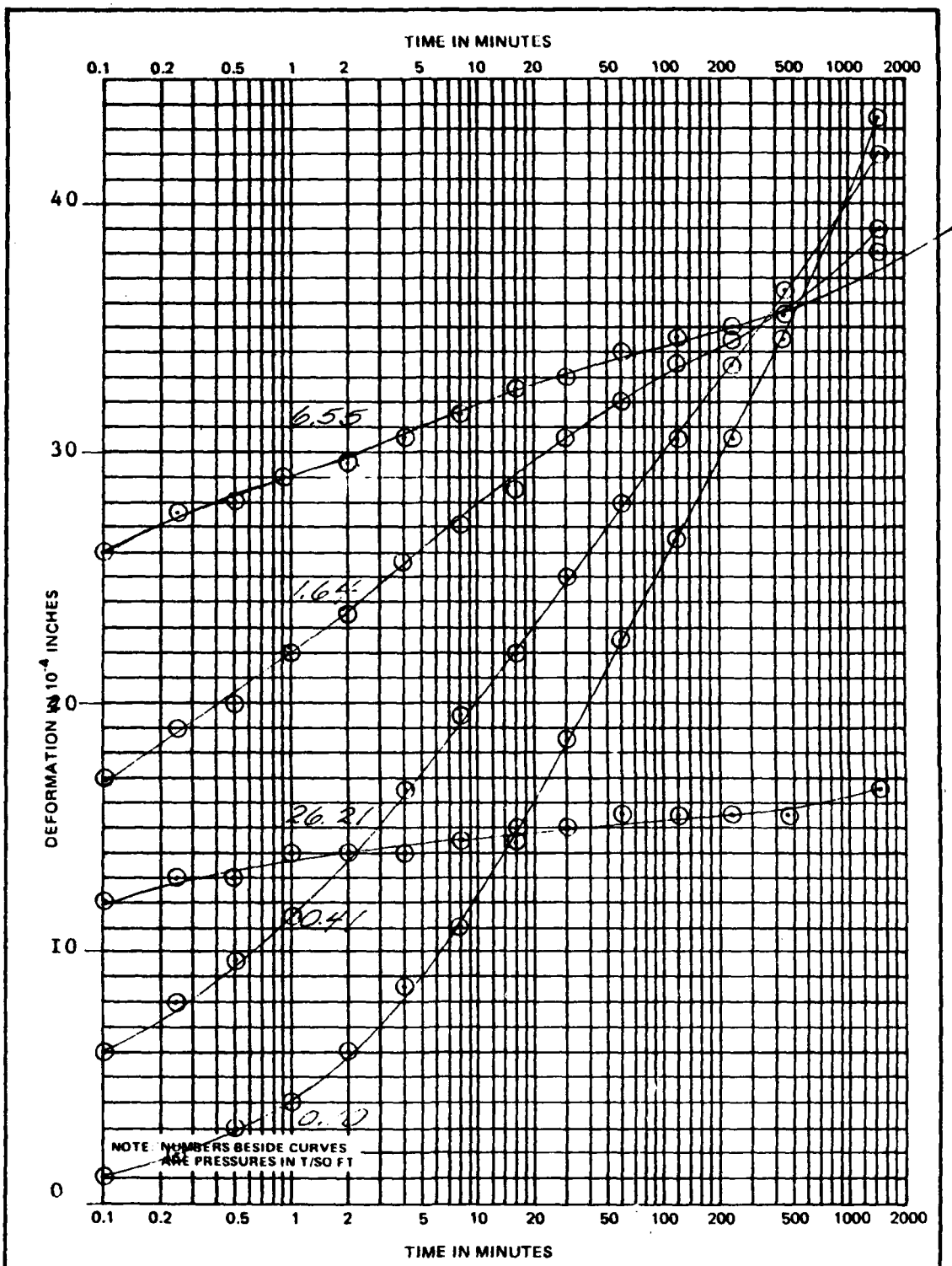
PLATE 92, PHASE IV



PROJECT Seven Oaks Dam, Div. No 100030			
AREA (Mill Creek-Impervious Borrow, Program 2: Phase IV)			
BORING NO CBT-86-30	SAMPLE NO Blend 2A	DEPTH EL 5.0'-11.0'	DATE Sept. 1987

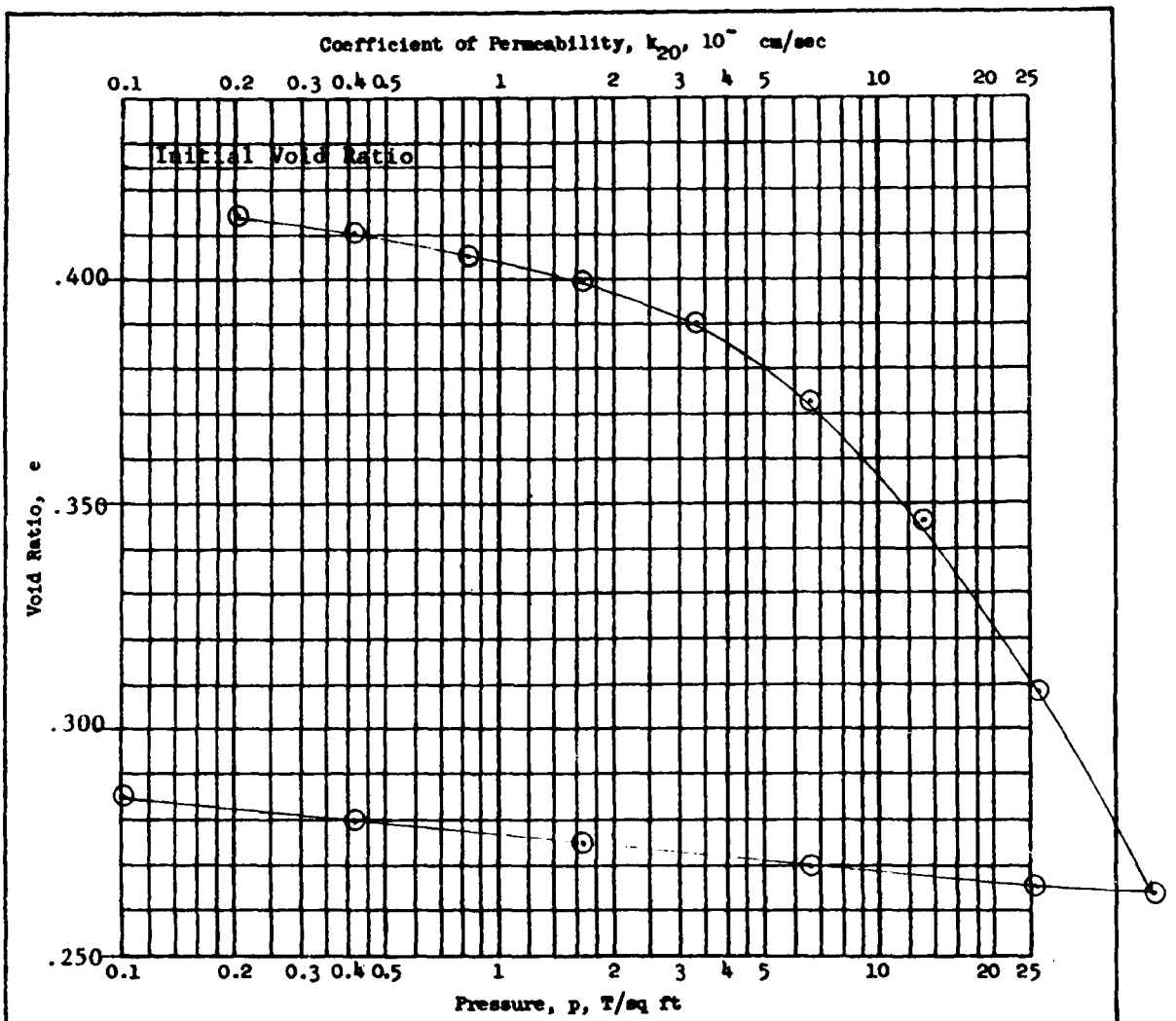
ENG FORM 1 MAY 83 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST–TIME CURVES** (TRANSLUCENT)

PLATE 93, PHASE IV



PROJECT Seven Oaks Dam, Div No. 100030			
AREA (Mill Creek-Impervious Borrow, Program 2: Phase IV)			
BORING NO CBT-86-30	SAMPLE NO B1and 2A	DEPTH EL 5.0'-11.0'	DATE Sept 1987
ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE	CONSOLIDATION TEST–TIME CURVES (TRANSLUCENT)		

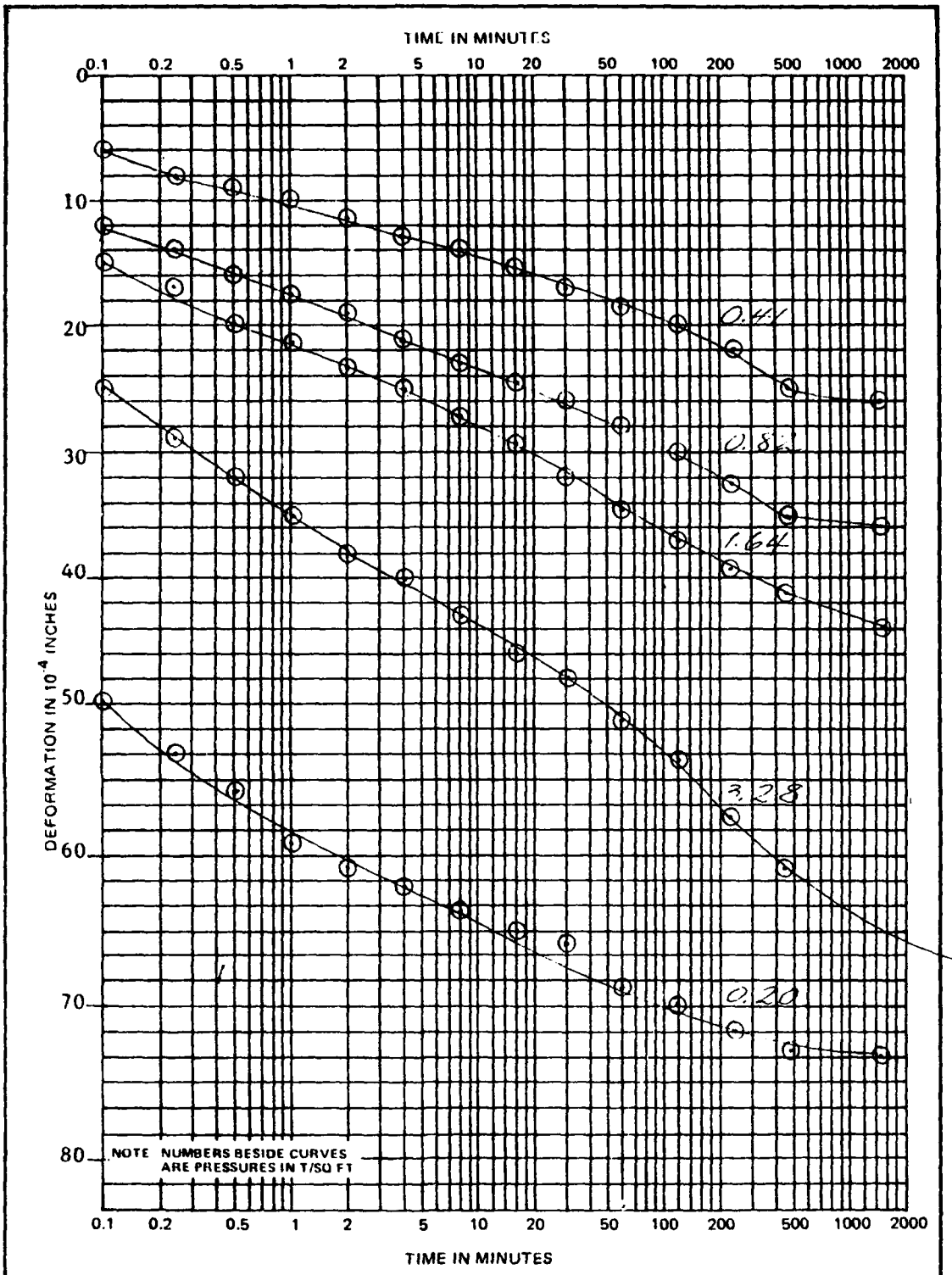
PLATE 94, PHASE IV



Type of Specimen Remolded			Before Test		After Test	
Diam 4.4 in.	Ht 1.0 in.		Water Content, w_o	14.3 %	w_f	10.8 %
Overburden Pressure, Po T/sq ft			Void Ratio, e_o	1425	e_f	.288
Preconsol. Pressure, p_c T/sq ft			Saturation, S_o	90.1 %	S_f	100 %
Compression Index, C_c			Dry Density, γ_d	116.9 lb/ft^3		
Classification Clayey Sand (SC)			k_{20} at e_o =	x 10^{-} cm/sec		
LL 33	G_s 2.67		Project SEVEN OAKS DAM			
PL 20	D_{10}		Mill Creek Impervious Borrow, Program 2			
Remarks Div. No. 100030			Area (Phase IV)			
98% maximum density at opt.			Boring No. CBT-86-30	Sample No. Blend 2A		
plus+2%			Depth El 5'-11'	Date Sept. 1987		
			CONSOLIDATION TEST REPORT			

ENG FORM 1 MAY 63 2090 PREVIOUS EDITIONS ARE OBSOLETE.

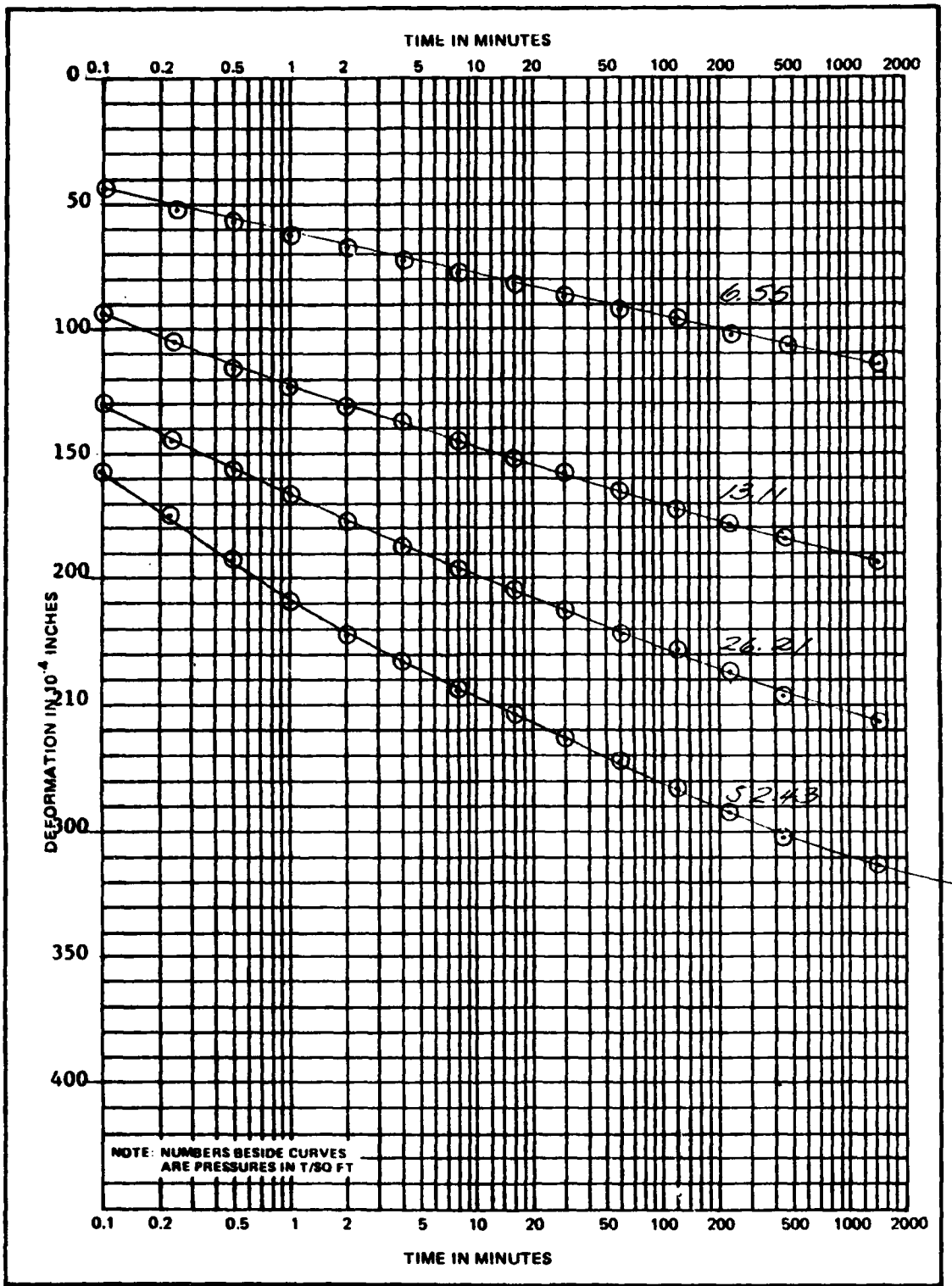
PLATE 95, PHASE IV



PROJECT	SEVEN OAKS DAM - Div. No. 100030			
AREA	(Mill Creek Impervious Borrow, Program 2, Phase IV)			
BORING NO.	CBT-86-30	SAMPLE NO. Blend 2A	DEPTH EL 5'-11'	DATE Sept. 1987

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST—TIME CURVES** (TRANSLUCENT)

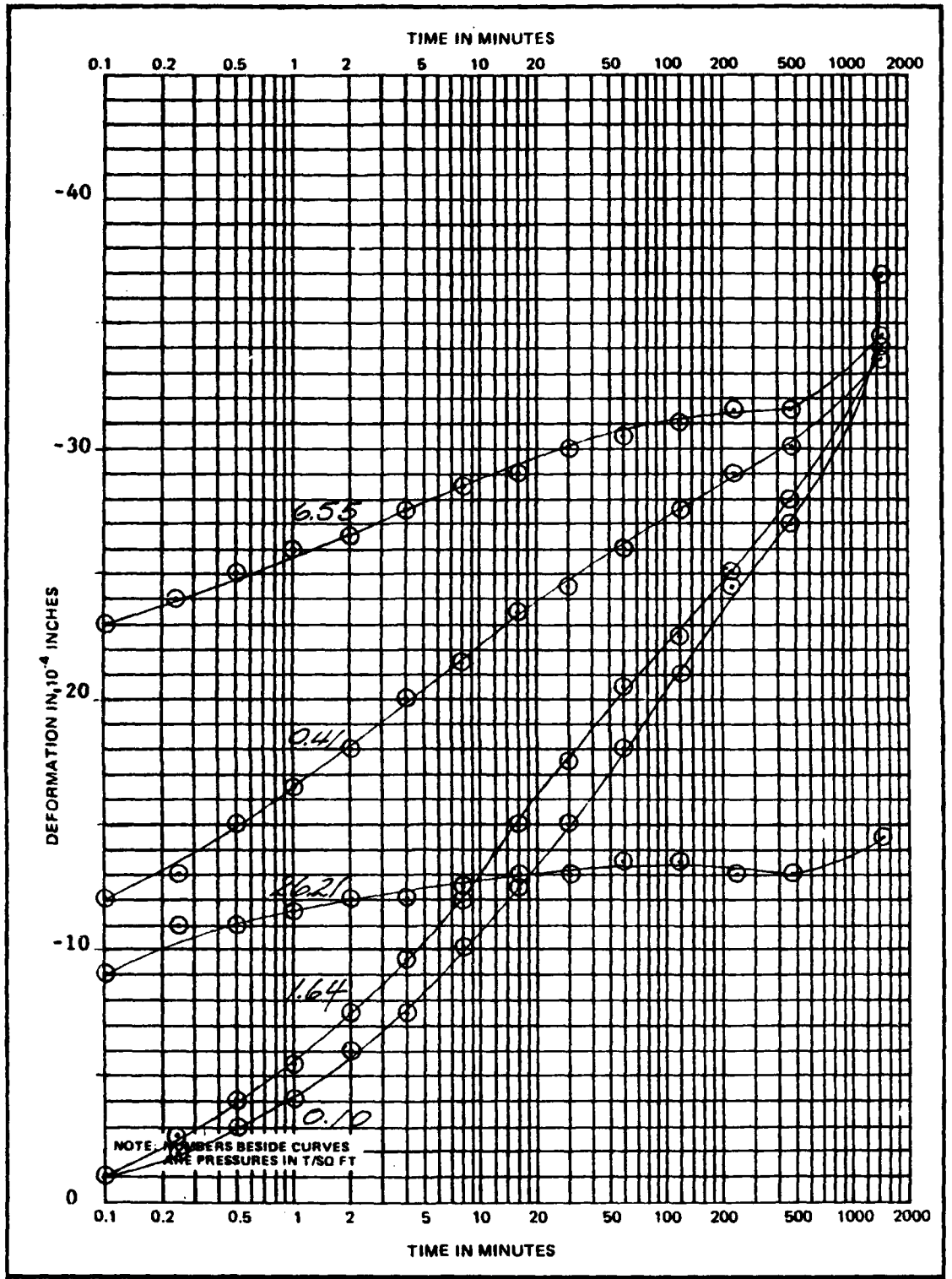
PLATE 96, PHASE IV



PROJECT	SEVEN OAKS DAM - Div. No. 100030			
AREA	(Mill Creek Impervious Borrow, Program 2, Phase IV)			
BORING NO.	CBT-86-30	SAMPLE NO Blend 2A	DEPTH EL 5'-11'	DATE Sept. 1987

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST—TIME CURVES** (TRANSLUCENT)

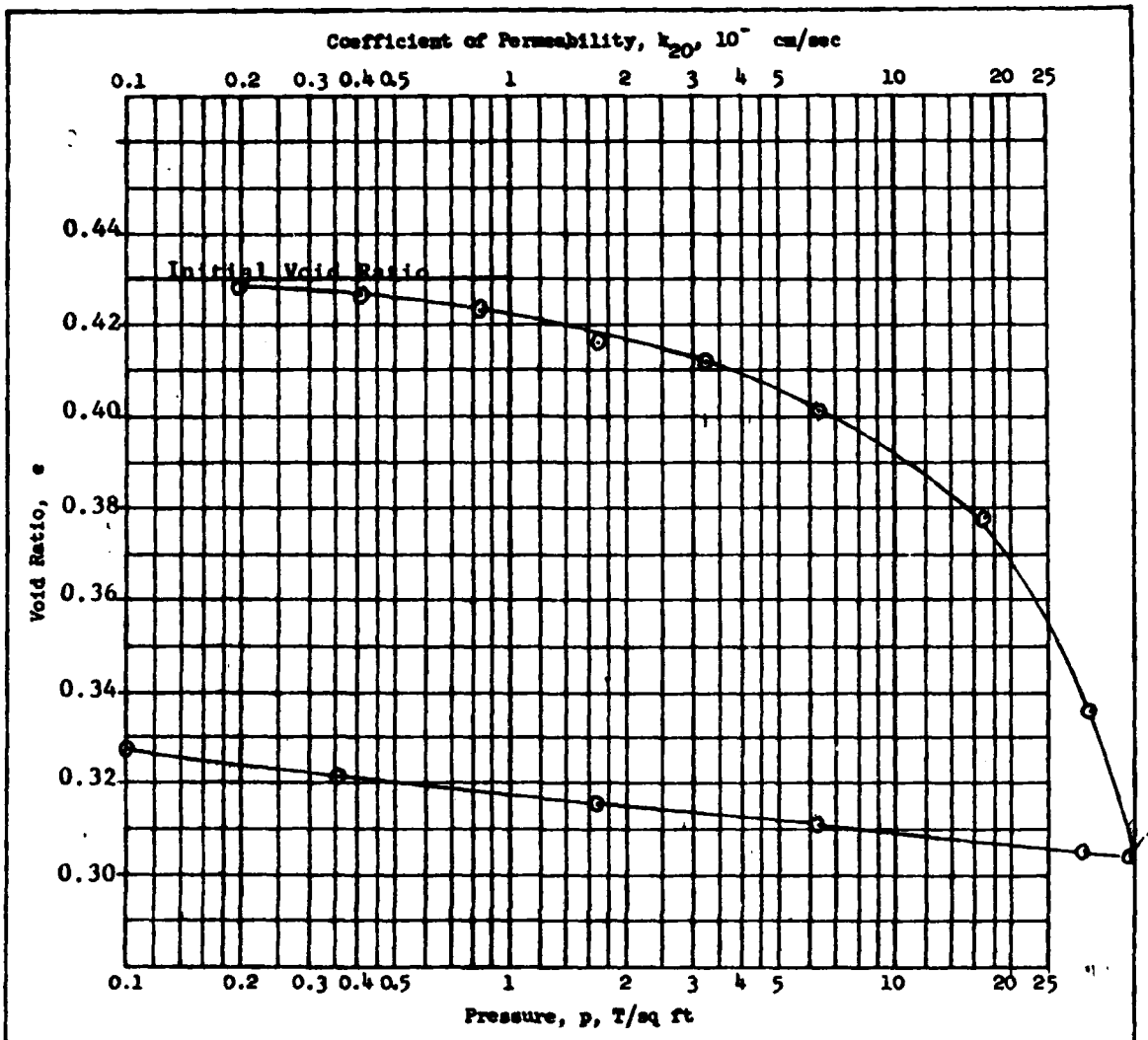
PLATE 57, PHASE IV



PROJECT	SEVEN OAKS DAM - Div. No. 100030			
AREA	(Mill Creek Impervious Borrow, Program 2, Phase IV)			
BORING NO.	CBT-86-30	SAMPLE NO. Blend 2A	DEPTH EL 5'-11'	DATE Sept. 1987

ENG FORM 1 MAY 83 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST--TIME CURVES** (TRANSLUCENT)

PLATE 98, PHASE IV

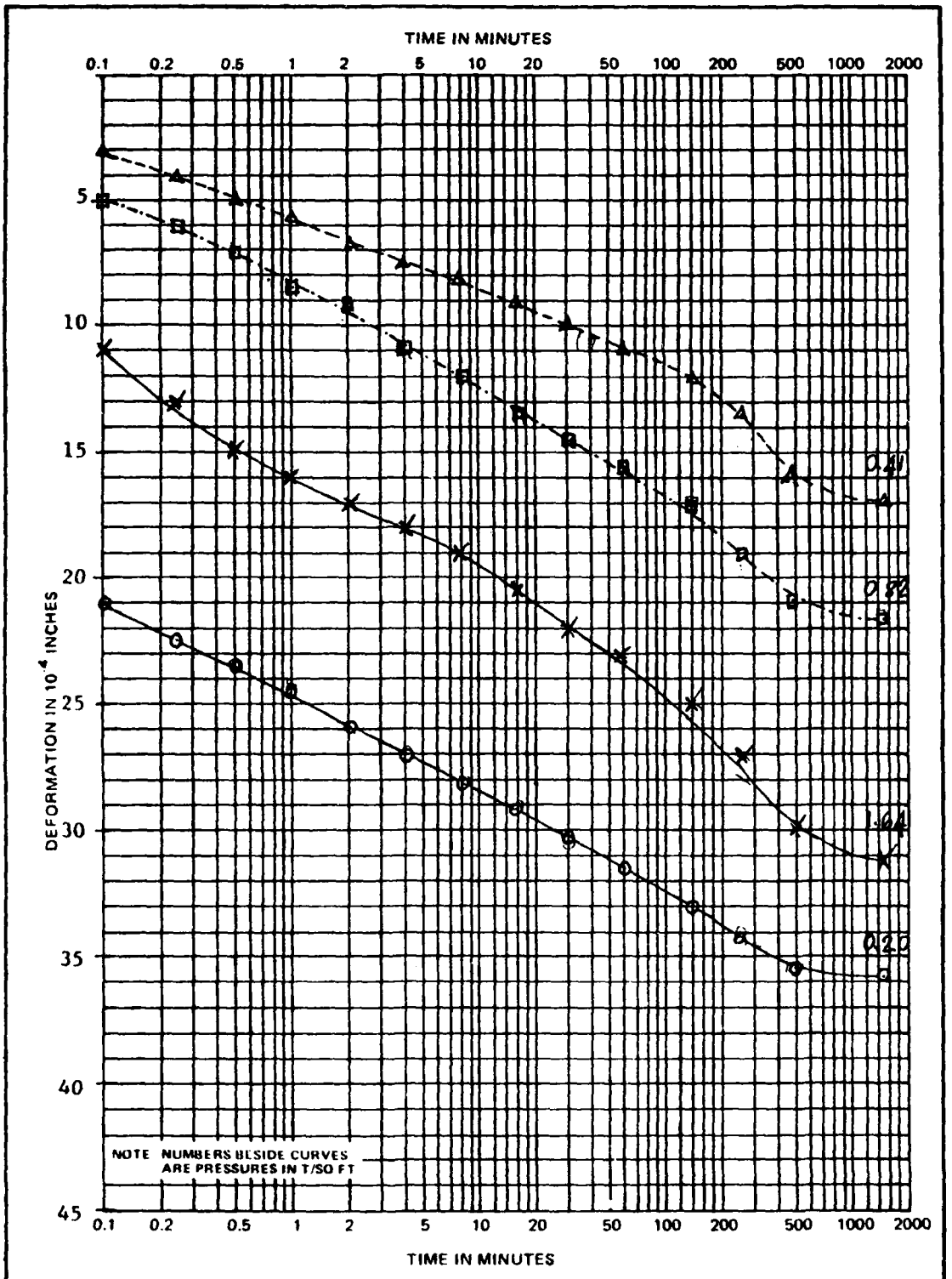


Type of Specimen	Remolded			Before Test		After Test	
Diam 4.4 in.	Ht 1.0 in.		Water Content, w_o	10.6 %	w_f	11.9 %
Overburden Pressure, Po	T/sq ft		Void Ratio, e_o	0.433	e_f	0.339
Preconsol. Pressure, P_c	T/sq ft		Saturation, S_o	65 %	S_f	94 %
Compression Index, C_c			Dry Density, γ_d	116.7 lb/ft^3		
Classification Clayey Sand (SC)			k_{20} at e_o =	x 10^{-} cm/sec		
LL 33	G_s 2.68		Project UPPER SANTA ANA RIVER			
PL 20	D_{10}		Seven Oaks Dam, Program 2			
Remarks Div. No. 100030			Area Mill Creek Impervious Borrow (Phase IV)			
98% maximum density at optimum -2%.			Boring No. CBT-86-30	Blend 2A		
			Depth El 5.0'-11.0'	Date Sept. 1987		
			CONSOLIDATION TEST REPORT			

ENG FORM 1 MAY 63 2090 PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT)

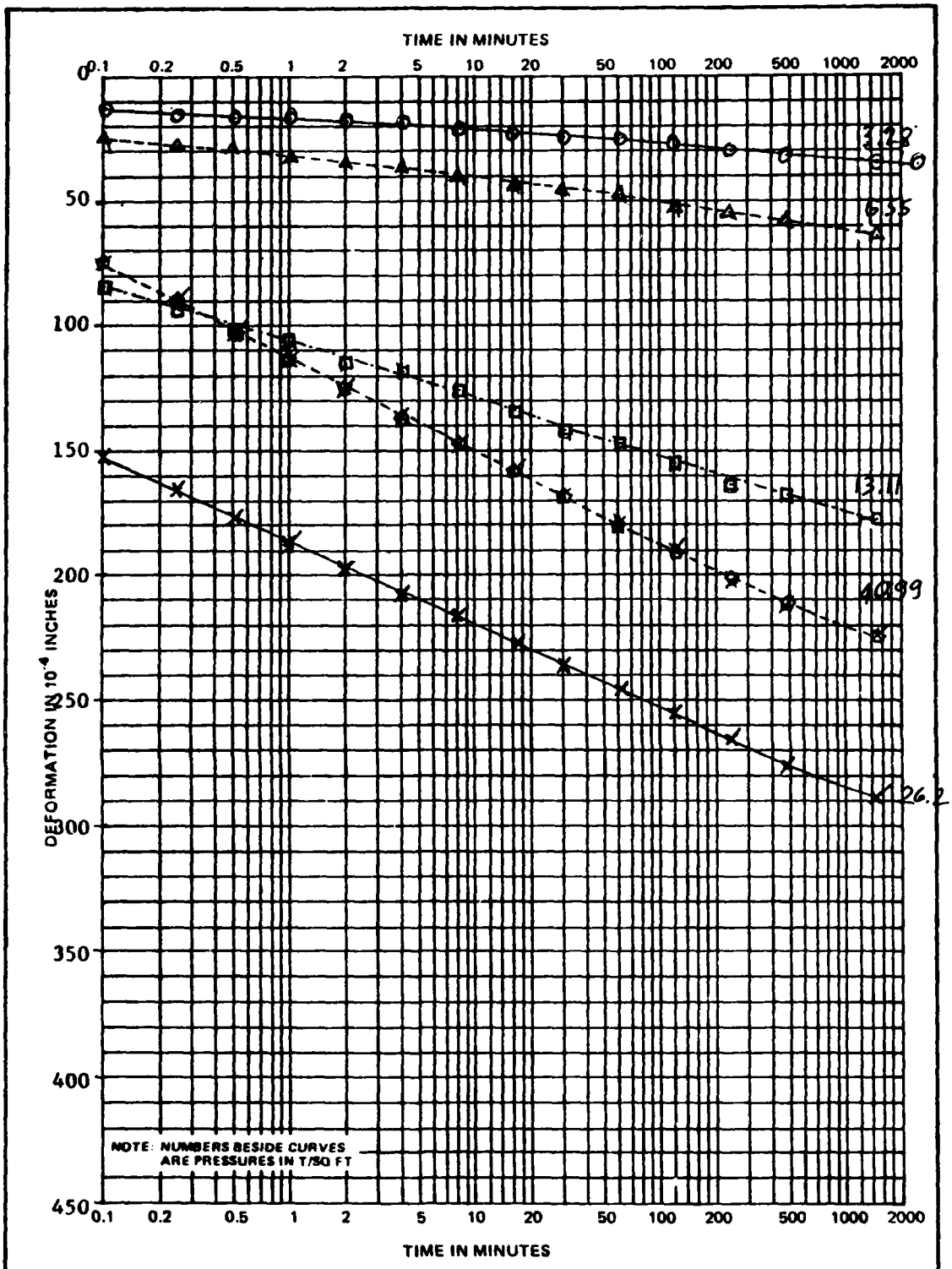
PLATE 99, PHASE IV

GPO 936-736



PROJECT	SEVEN OAKS DAM - Div. No. 100030			
AREA	(Mill Creek Impervious Borrow, Program 2, Phase IV)			
BORING NO. CBT-86-30	SAMPLE NO Blend 2A	DEPTH EL 5.0'-11.0'	DATE Sept. 1987	
ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE	**CONSOLIDATION TEST--TIME CURVES** (TRANSLUCENT)			

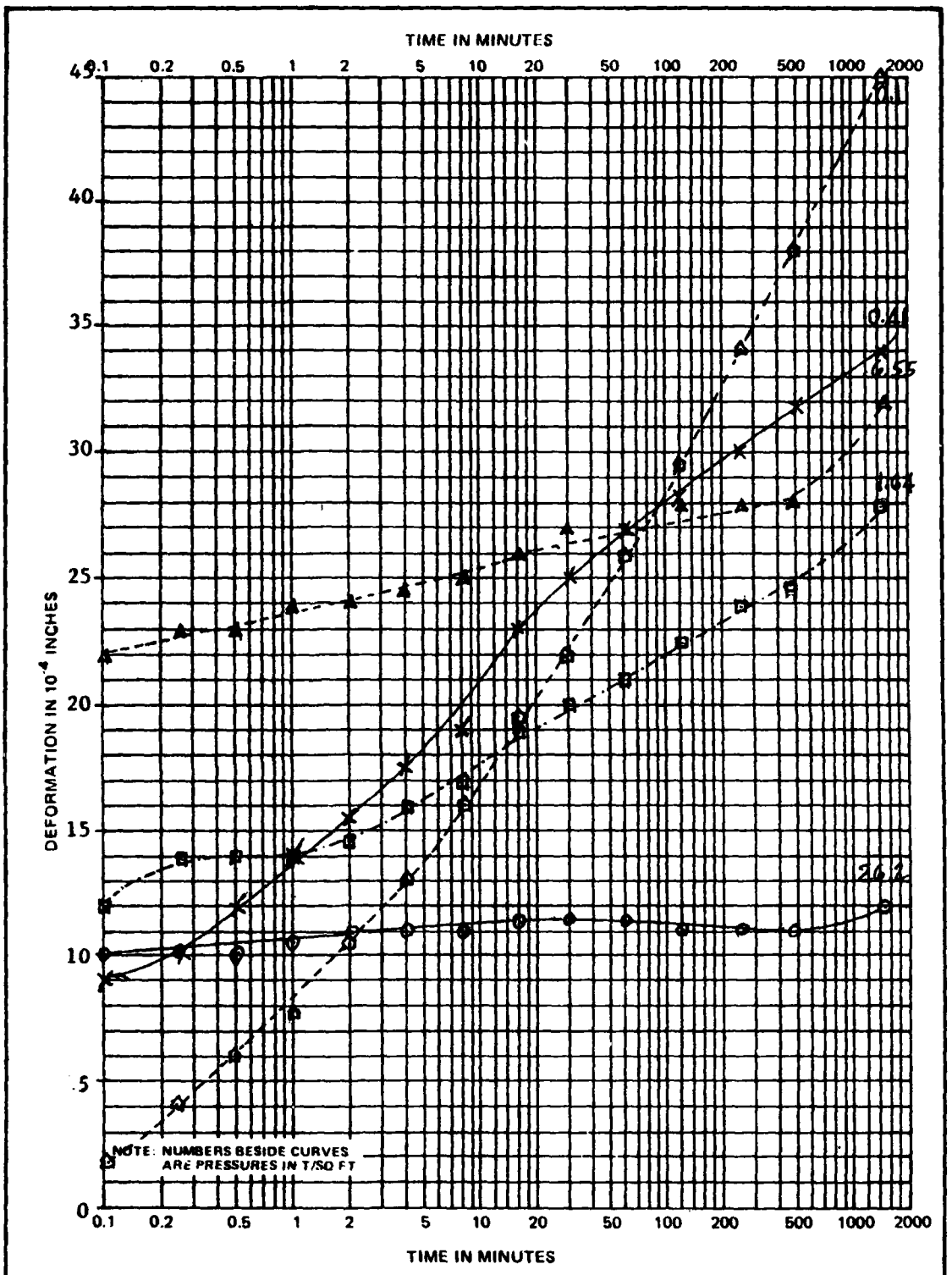
PLATE 10a PHASE IV



PROJECT	SEVEN OAKS DAM - Div. No. 100030				
AREA	(Mill Creek Impervious Borrow, Program 2, Phase IV)				
BORING NO.	CBT-86-30	SAMPLE NO	Blend 2A	DEPTH EL	5.0'-11.0'
DATE	Sept. 1987				

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST–TIME CURVES** (TRANSLUCENT)

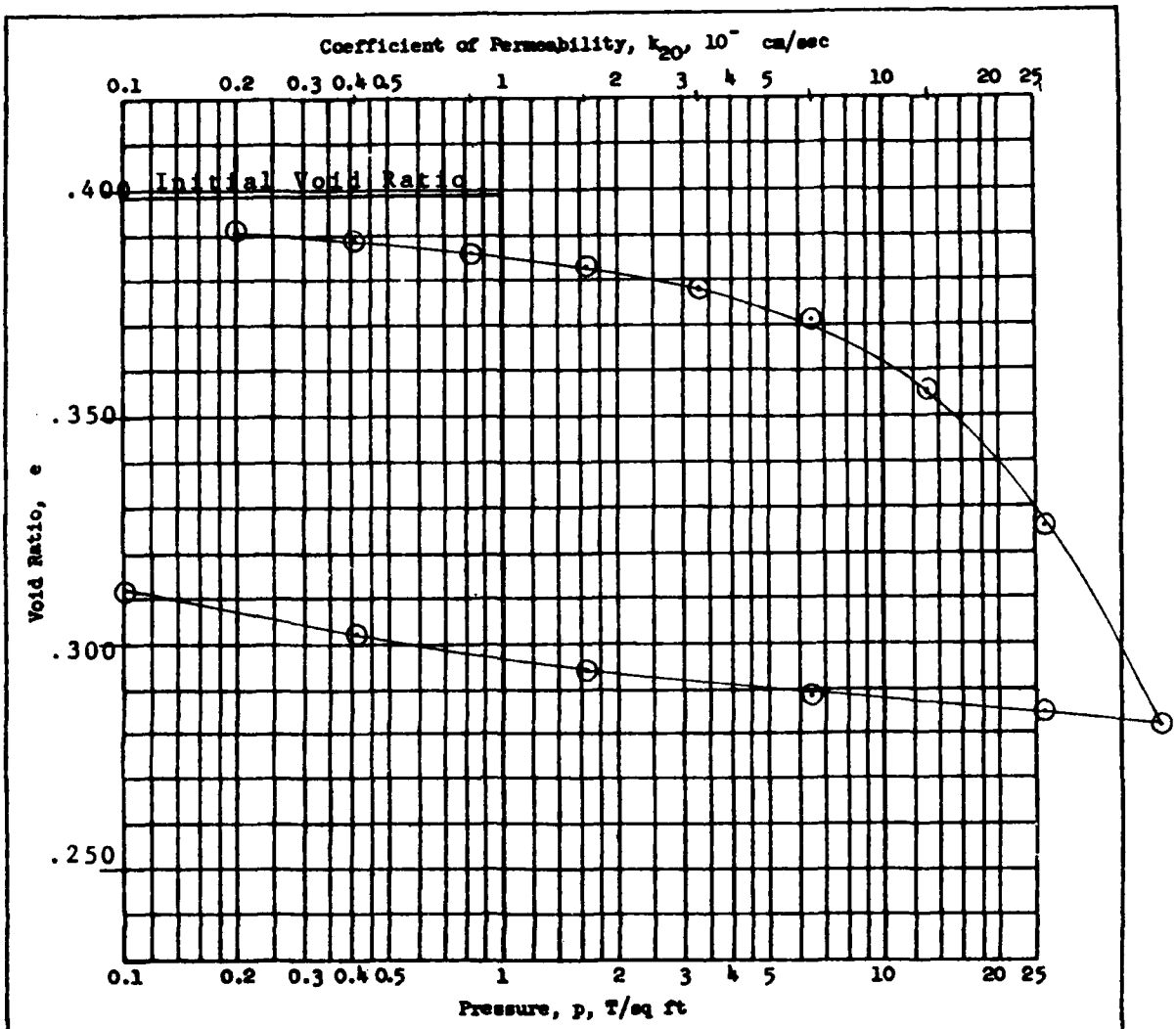
PLATE 101, PHASE IV



PROJECT	SEVEN OAKS DAM - Div. No. 100030			
AREA	(Mill Creek Impervious Borrow, Program 2, Phase IV)			
BORING NO.	CBT-86-30	SAMPLE NO Blend 2A	DEPTH EL 5.0'-11.0'	DATE Sept. 1987

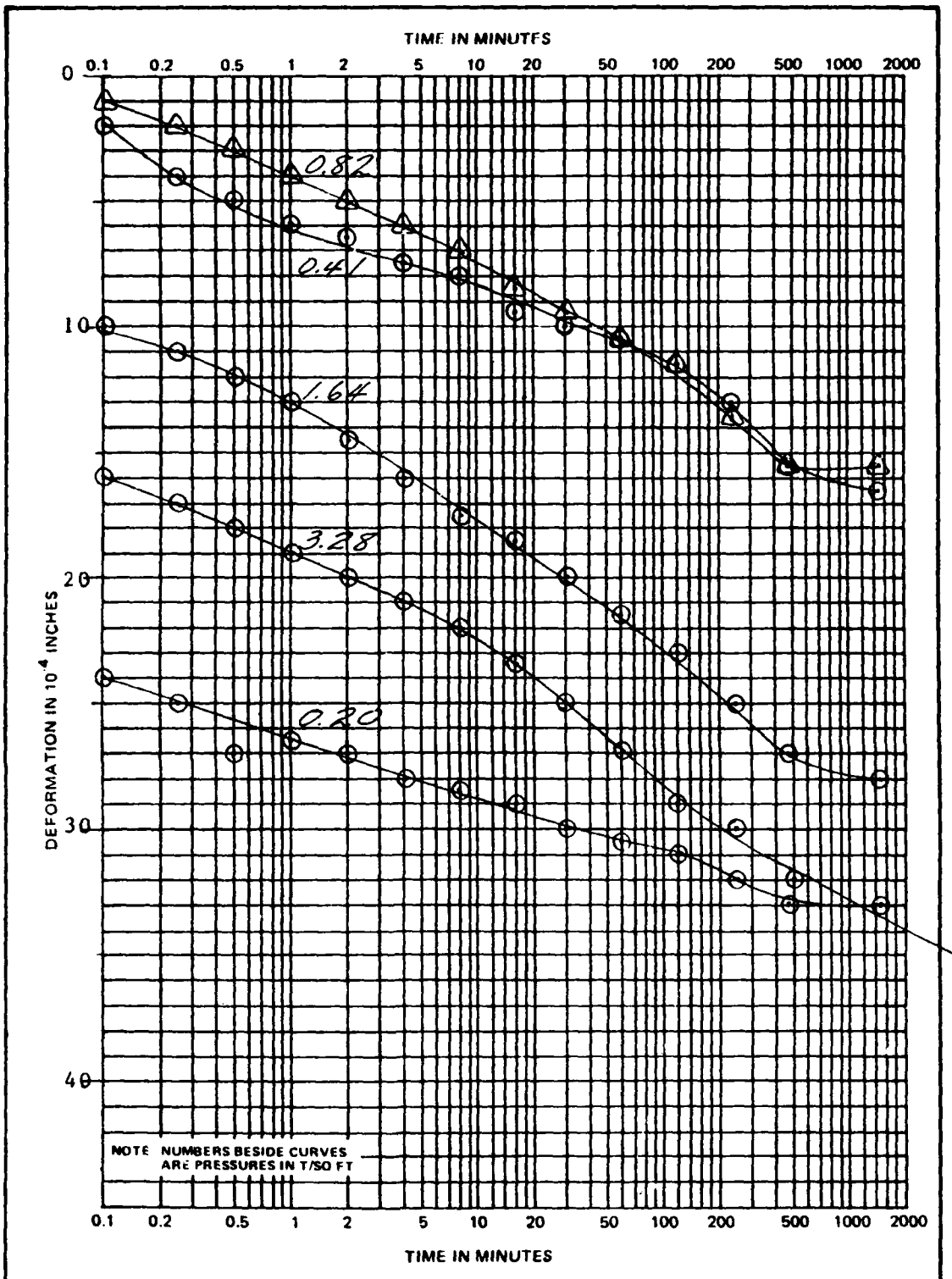
ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST–TIME CURVES** (TRANSLUCENT)

PLATE 102, PHASE IV



Type of Specimen Remolded		Before Test		After Test	
Diam 4.4 in.	Ht 1.0 in.	Water Content, w_o	11.6 %	w_f	11.0 %
Overburden Pressure, Po T/sq ft		Void Ratio, e_o	0.399	e_f	0.294
Preconsol. Pressure, Pc T/sq ft		Saturation, S_o	78 %	S_f	100 %
Compression Index, C_c		Dry Density, γ_d	119.5 lb/ft^3		
Classification Clayey Sand(SC-SM)		k_{20} at e_o =	x 10^{-} cm/sec		
LL 22	G_s 2.73	Project Seven Oaks Dam			
PL 16	D_{10}	Mill Creek Jmpervious Borrow			
Remarks Div.No. 100035		Area (Phase IV,Program 2)			
Remolded to 98% maximum		Boring No. CBH 86-29		Sample No. Blend 7A	
density at optimum.		Depth El 0.0'-10.0'		Date October1987	
		CONSOLIDATION TEST REPORT			

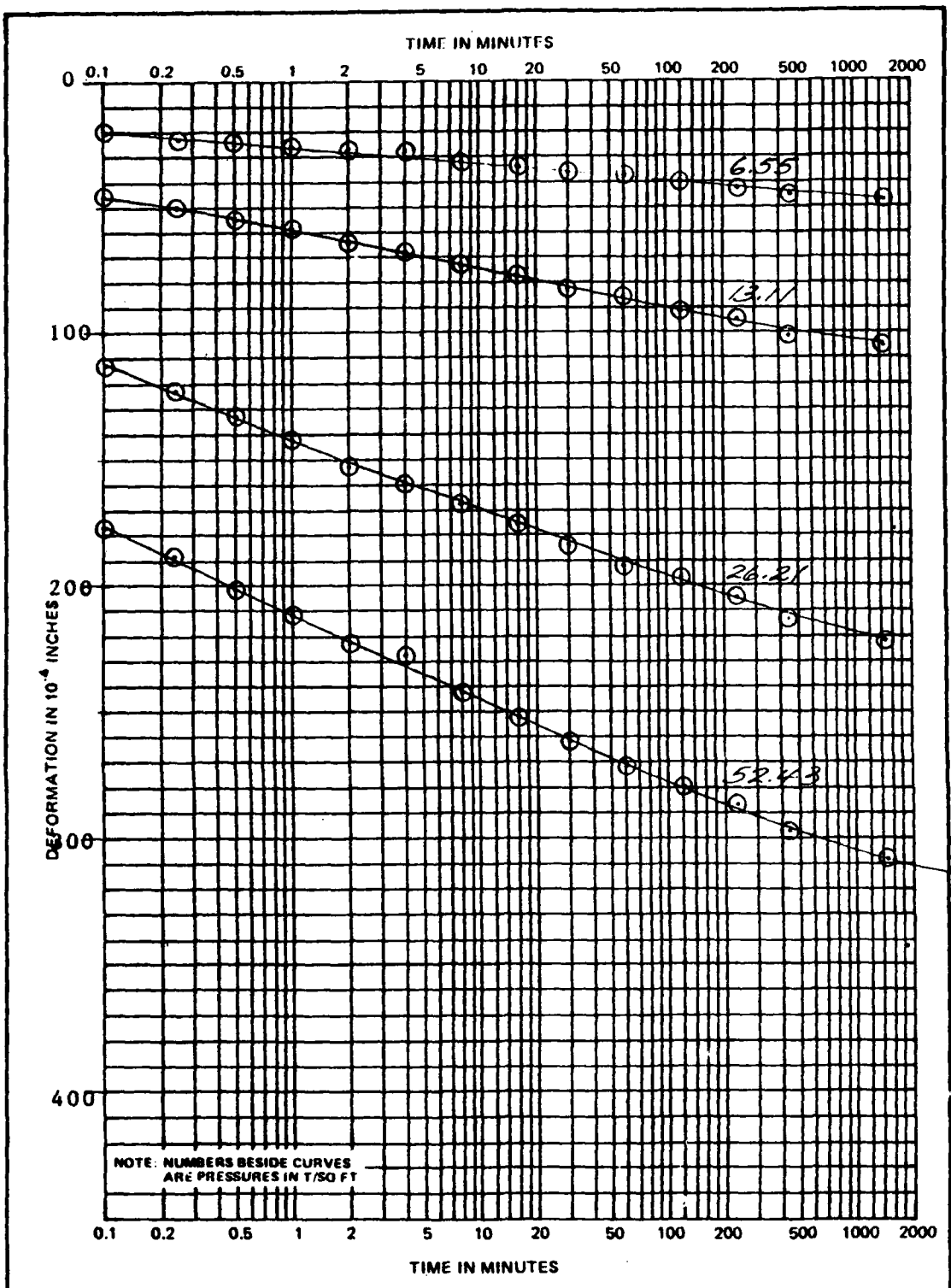
ENG FORM 1 MAY 69 2090 PREVIOUS EDITIONS ARE OBSOLETE.



PROJECT Seven Oaks Dam - Mill creek Impervious Borrow			
AREA Div.No. 100035 (Phase IV, Program 2)			
BORING NO CBH 86-29	SAMPLE NO Blend 7A	DEPTH EL 0.0'-10.0'	DATE October 1987

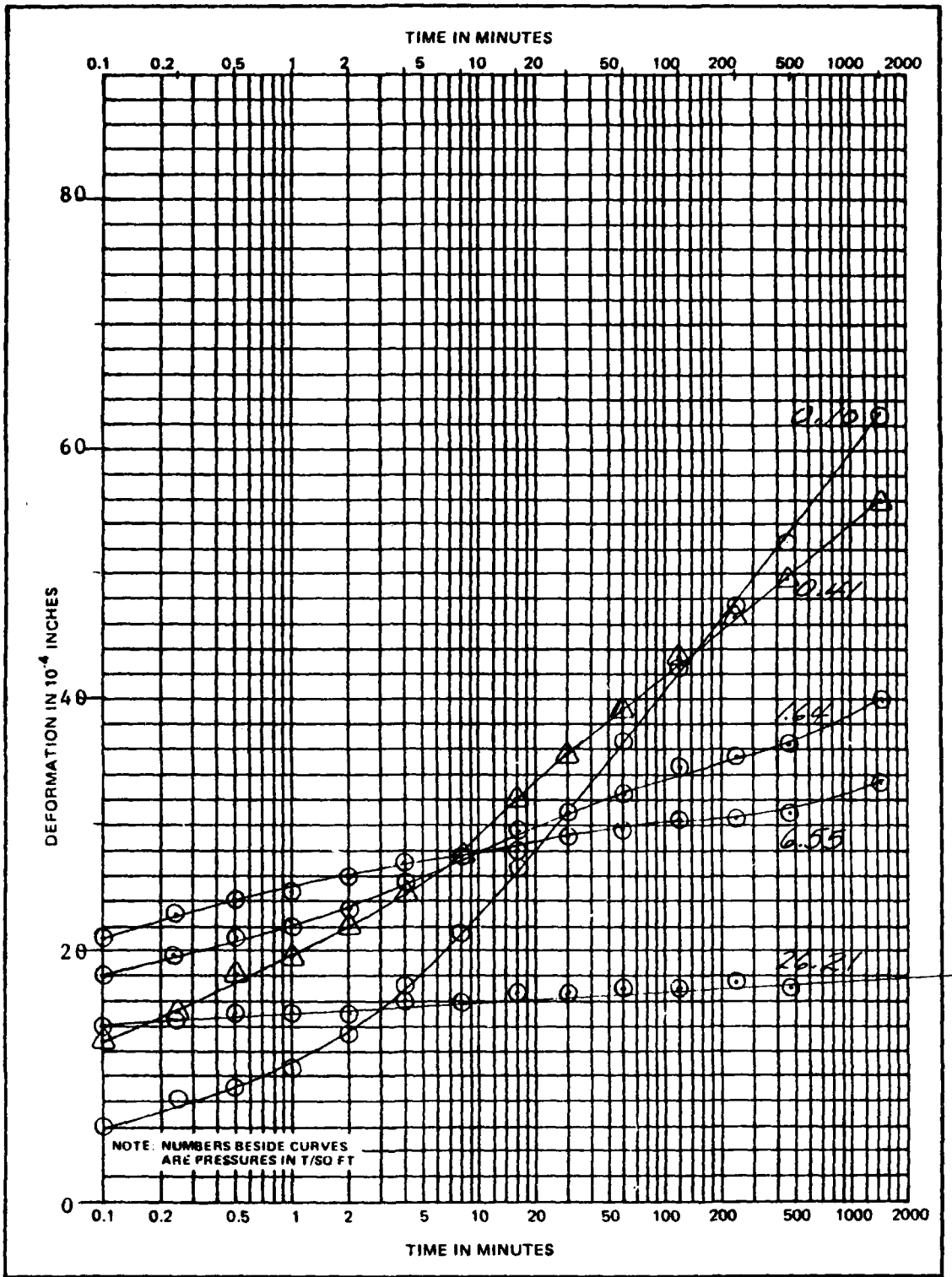
ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST–TIME CURVES** (TRANSLUCENT)

PLATE 104 PHASE IV



PROJECT Seven Oaks Dam - Mill Creek Impervious Borrow			
AREA Div.No. 100035 (Phase IV, Program 2)			
BORING NO. CBH 86-29	SAMPLE NO. Blend 7A	DEPTH EL 0.0'-10.0'	DATE October 1987
ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE	**CONSOLIDATION TEST–TIME CURVES** (TRANSLUCENT)		

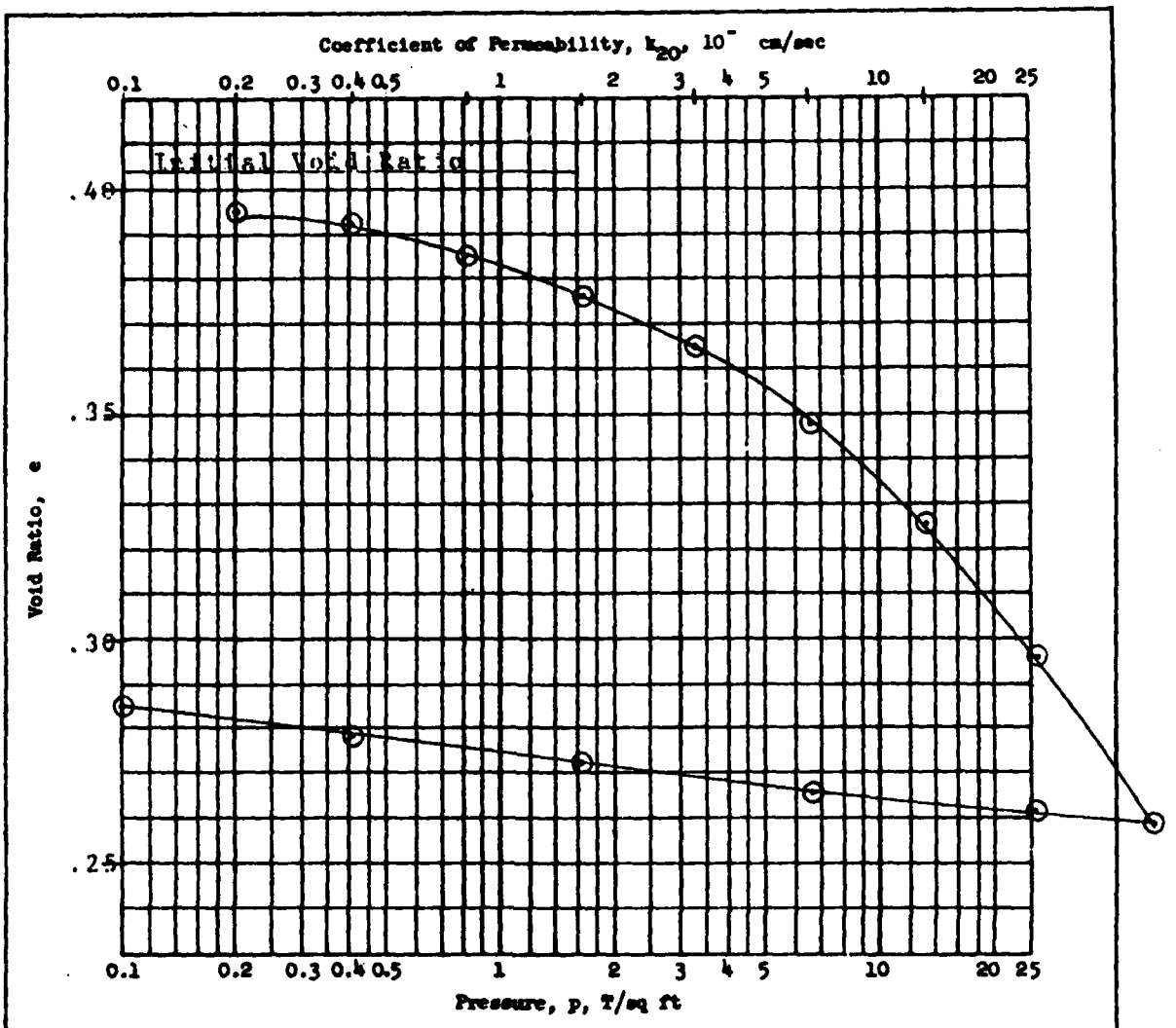
PLATE 105, PHASE IV



PROJECT Seven Oaks Dam - Mill Creek Impervious Borrow			
AREA Div.No. 100035 (Phase IV, Program 2)			
BORING NO. BH 86-29	SAMPLE NO. Blend 7A	DEPTH EL 0.0'-10.0'	DATE October 1987

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST–TIME CURVES** (TRANSLUCENT)

PLATE 106, PHASE IV

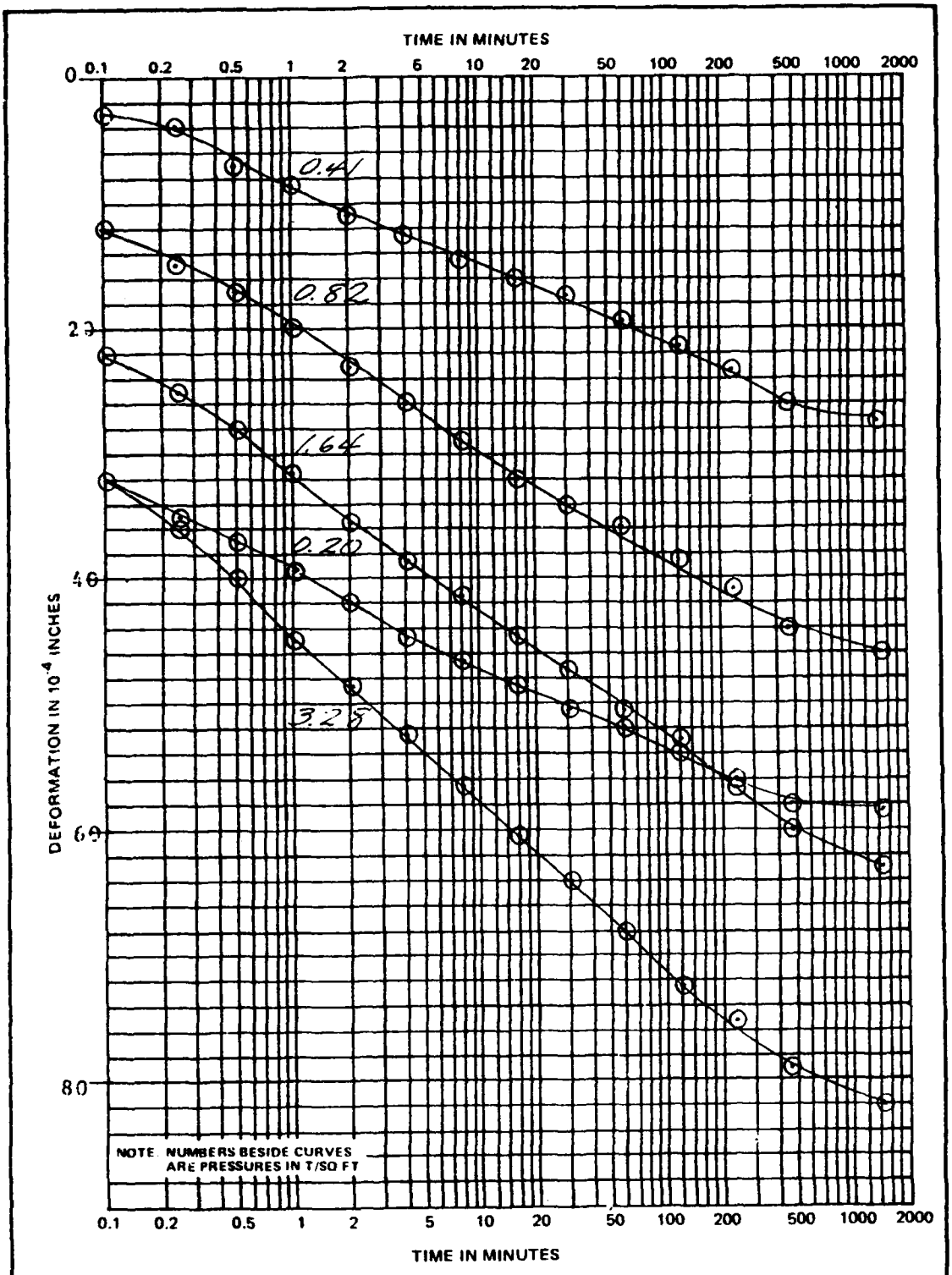


Type of Specimen Remolded		Before Test		After Test	
Diam 4.4 in.	Ht 1.0 in.	Water Content, w_o	14.0 %	w_f	10.6 %
Overburden Pressure, p_o T/sq ft		Void Ratio, e_o	0.404	e_f	0.283
Preconsol. Pressure, p_c T/sq ft		Saturation, S_o	93 %	S_f	100 %
Compression Index, C_c		Dry Density, γ_d	119.1 lb/ft^3		
Classification Clayey Sand(SC-SM)		k_{20} at e_o =	x 10^{-} cm/sec		
LL 22	G_s 2.73	Project Seven Oaks Dam			
PL 16	D_{10}	Mill Creek Impervious Borrow			
Remarks Div.No. 100035		Area (phase IV, Program 2)			
Remolded to 98% maximum		Boring No. CBH 86-29	Sample No. Blend 7A		
deusity at optimum +2%.		Depth El 0.0'10.0'	Date Oct. 1987		

CONSOLIDATION TEST REPORT

ENG FORM 1 MAY 63 2090 PREVIOUS EDITIONS ARE OBSOLETE.

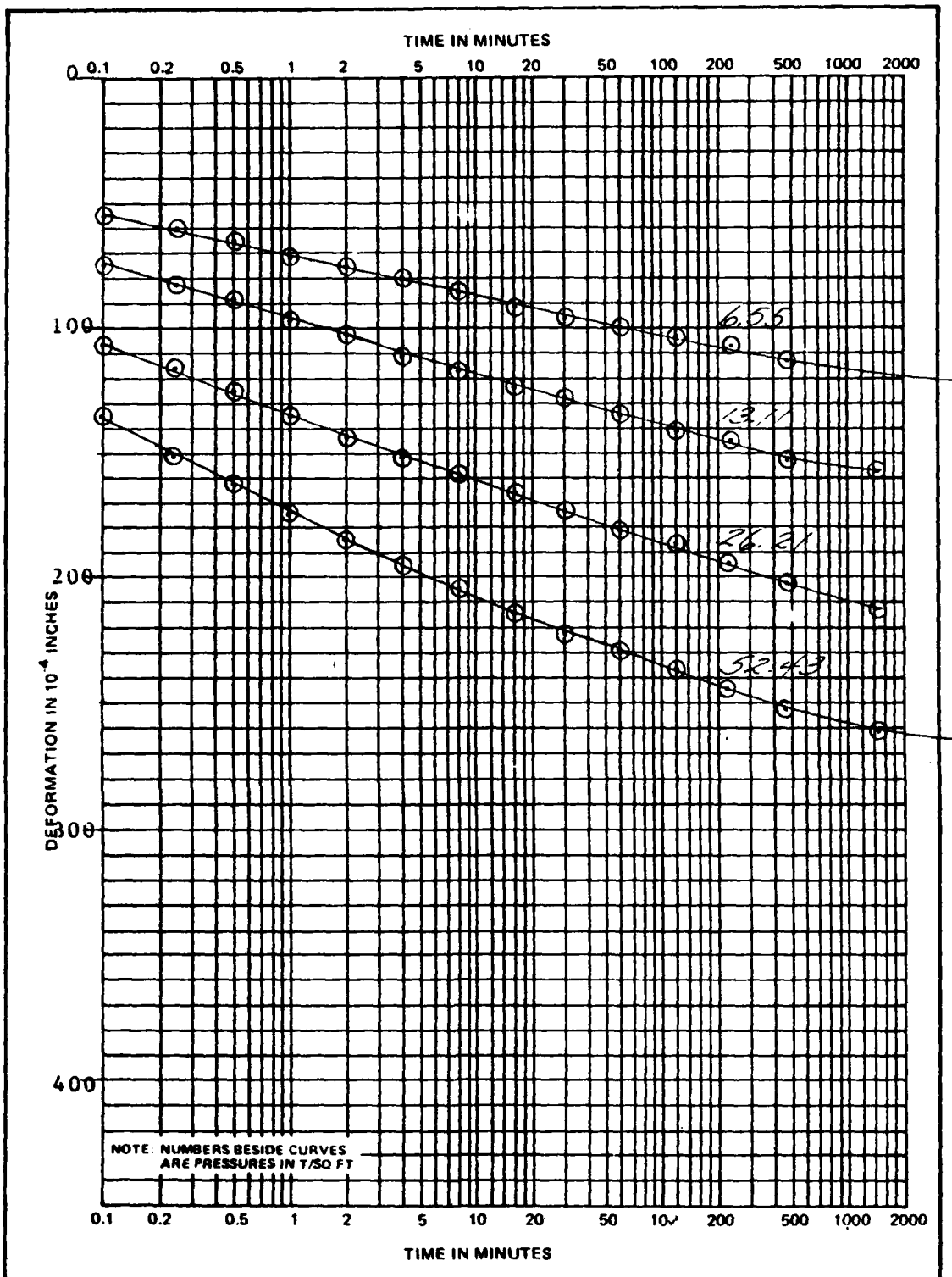
PLATE 07, PHASE IV



PROJECT Seven Oaks Dam - Mill Creek Impervious Borrow			
AREA Div.No. 100035 (Phase IV, Program 2)			
BORING NO. CBH 86-29	SAMPLE NO. Blend7A	DEPTH EL 0.0'-10.0'	DATE Oct. 1987

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST--TIME CURVES** (TRANSLUCENT)

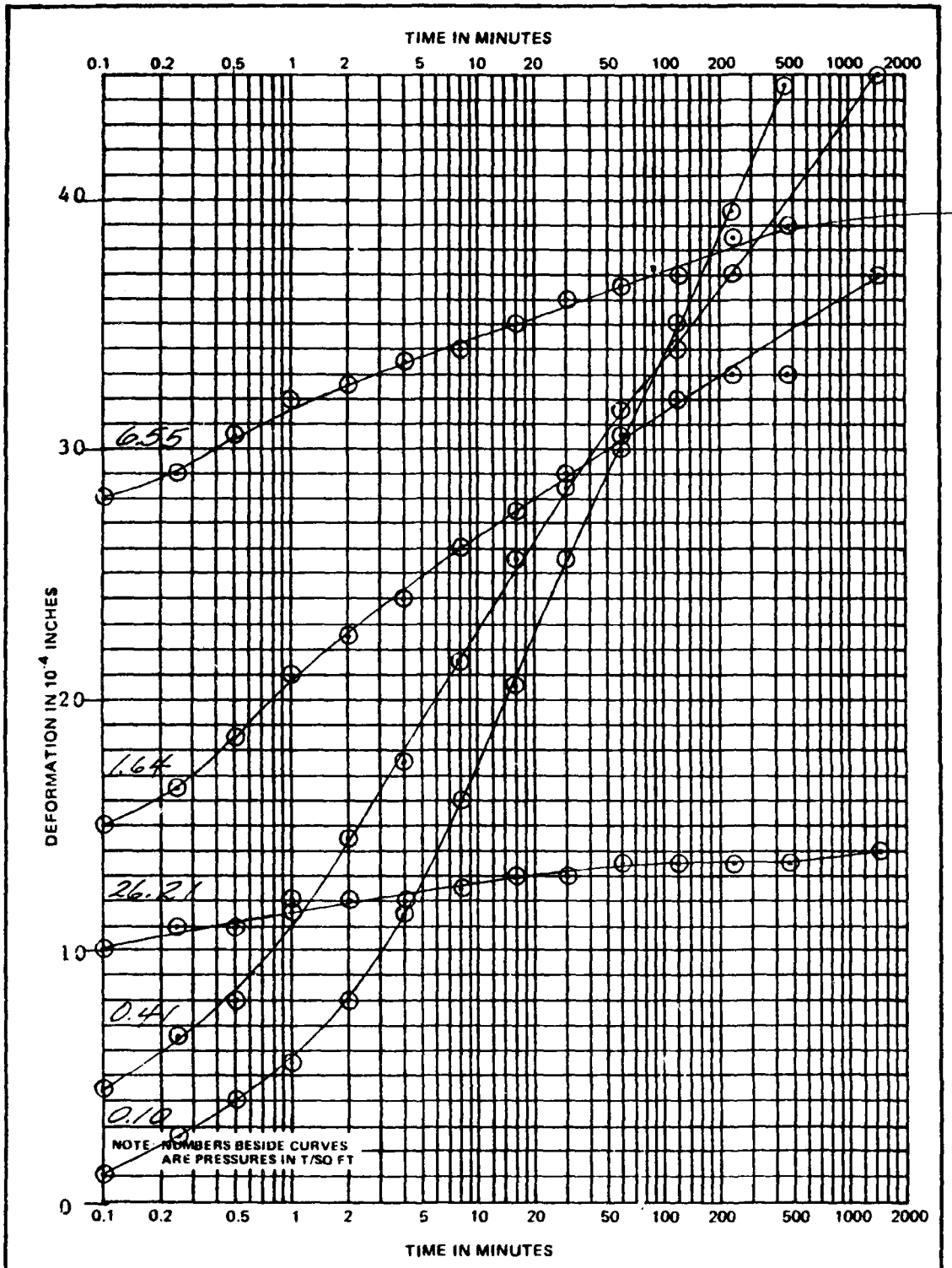
PLATE 108, PHASE IV



PROJECT	Seven Oaks Dam - Mill Creek Impervious Borrow			
AREA	Div.No, 100035 (Phase IV Program 2)			
BORING NO.	CBH 86-29	SAMPLE NO.	Blend 7A	DEPTH EL 0.0'-10.0'
DATE	Oct. 1987			

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST–TIME CURVES** (TRANSLUCENT)

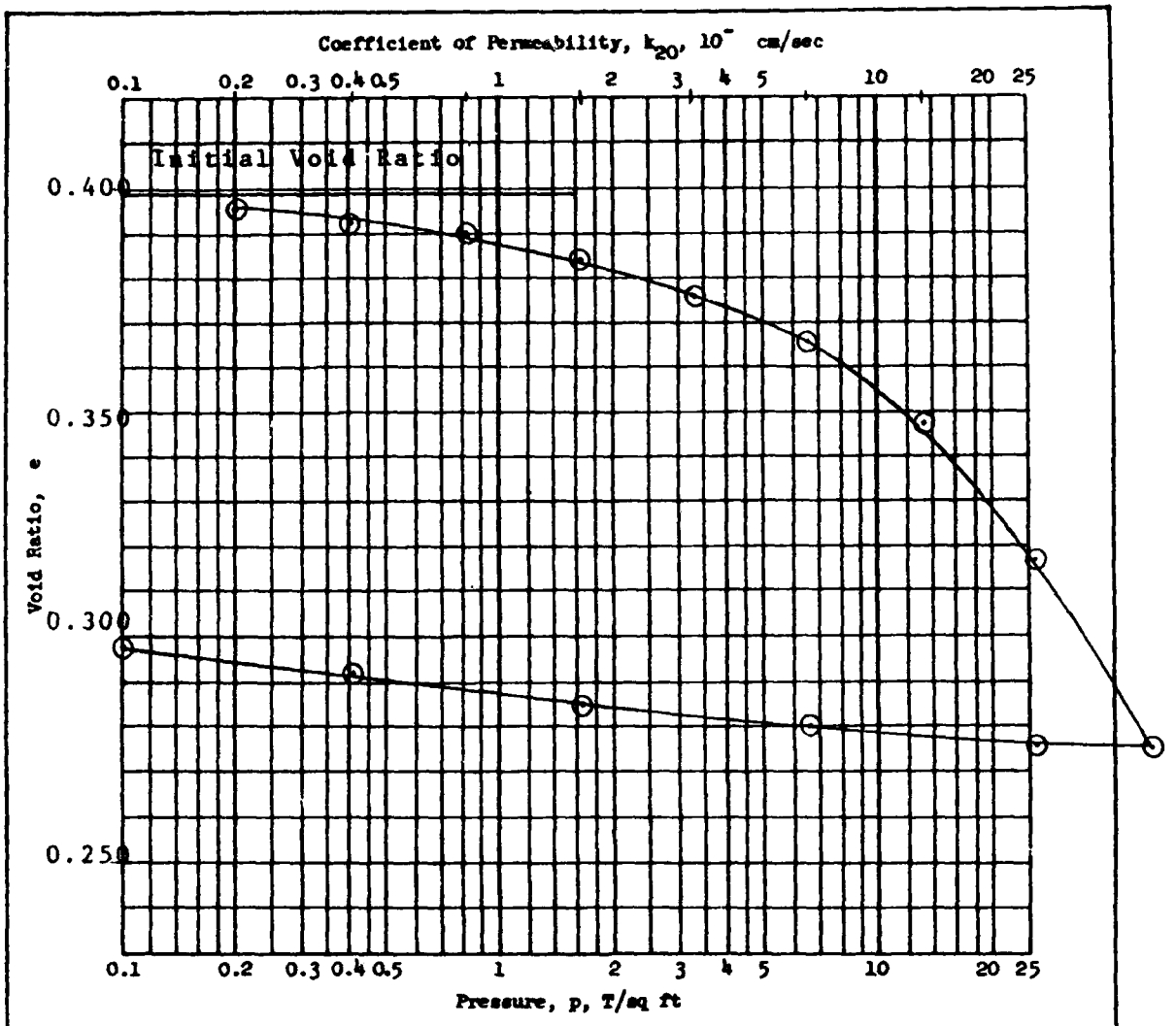
PLATE 109, PHASE IV



PROJECT	Seven Oaks dam - Mill Creek Impervious Barrow		
AREA	Div.No. 100035 (Phase IV, Program 2)		
BORING NO. CBH 86-29	SAMPLE NO Blend 7A	DEPTH EL 0.0'-10.0'	DATE Oct. 1987

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST–TIME CURVES** (TRANSLUCENT)

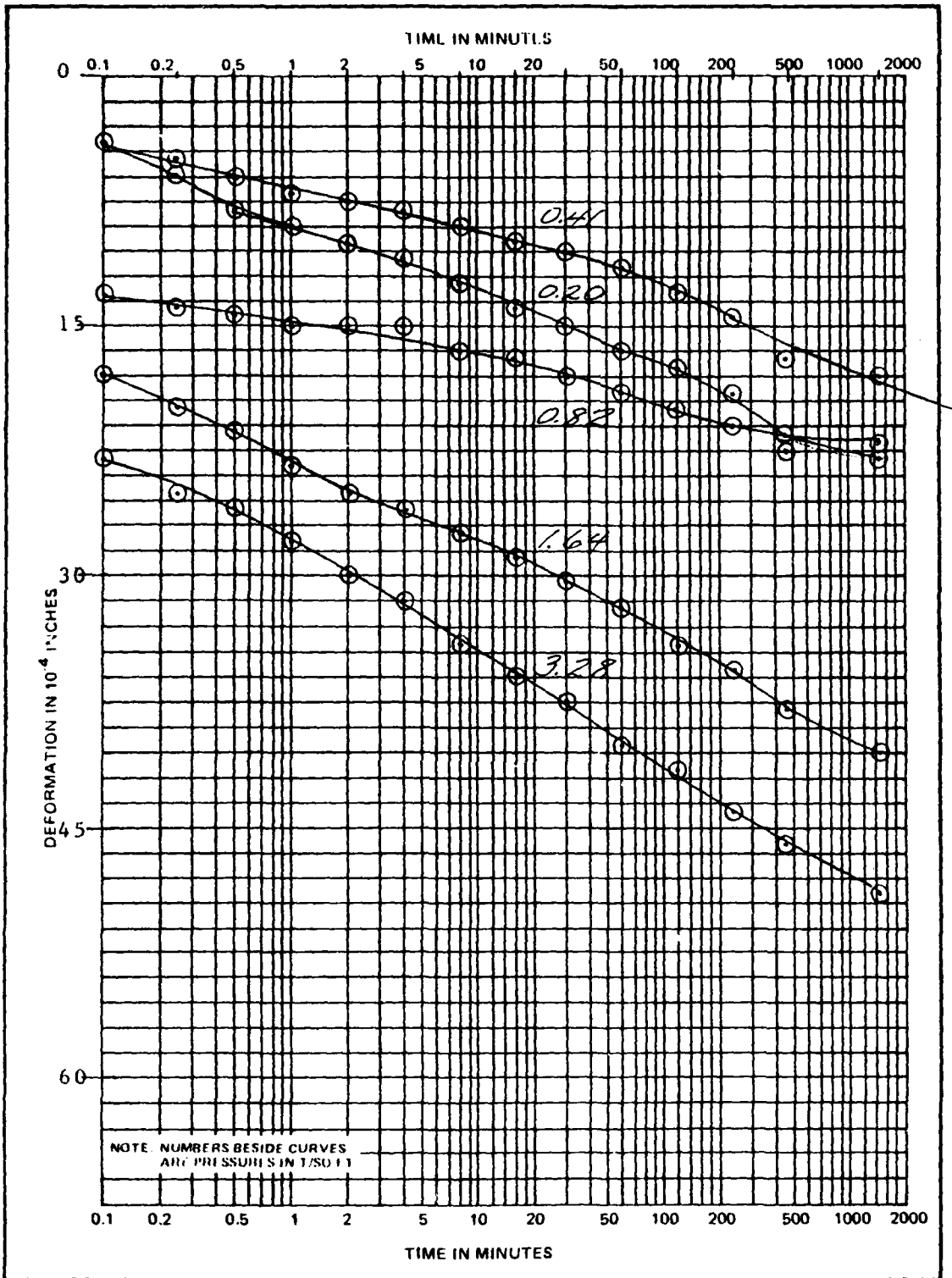
PLATE 10 , PHASE IV



Type of Specimen Remolded		Before Test		After Test	
Diam 4.4 in.	Ht 1.0 in.	Water Content, w_o	9.4 %	w_f	11.3 %
Overburden Pressure, p_o T/sq ft		Void Ratio, e_o	0.395	e_f	0.327
Preconsol. Pressure, p_c T/sq ft		Saturation, S_o	63.9 %	S_f	92.8 %
Compression Index, C_c		Dry Density, γ_d	119. lb/ft^3		
Classification Clayey Sand(SC-SM)		k_{20} at e_o =	x 10^{-} cm/sec		
LL 22	G_s 2.73	Project Seven Oaks Dam			
PL 16	D_{10}	Mill Creek Impervious Borrow			
Remarks Div.No.100035		Area (Phase IV, Program 2)			
Remolded to 98% maximum		Boring No. CBH 86-29	Sample No. Blend 7A		
density at optimum -2%.		Depth El 0.0'-10.0'	Date October1987		
		CONSOLIDATION TEST REPORT			

ENG FORM 1 MAY 69 2090 PREVIOUS EDITIONS ARE OBSOLETE.

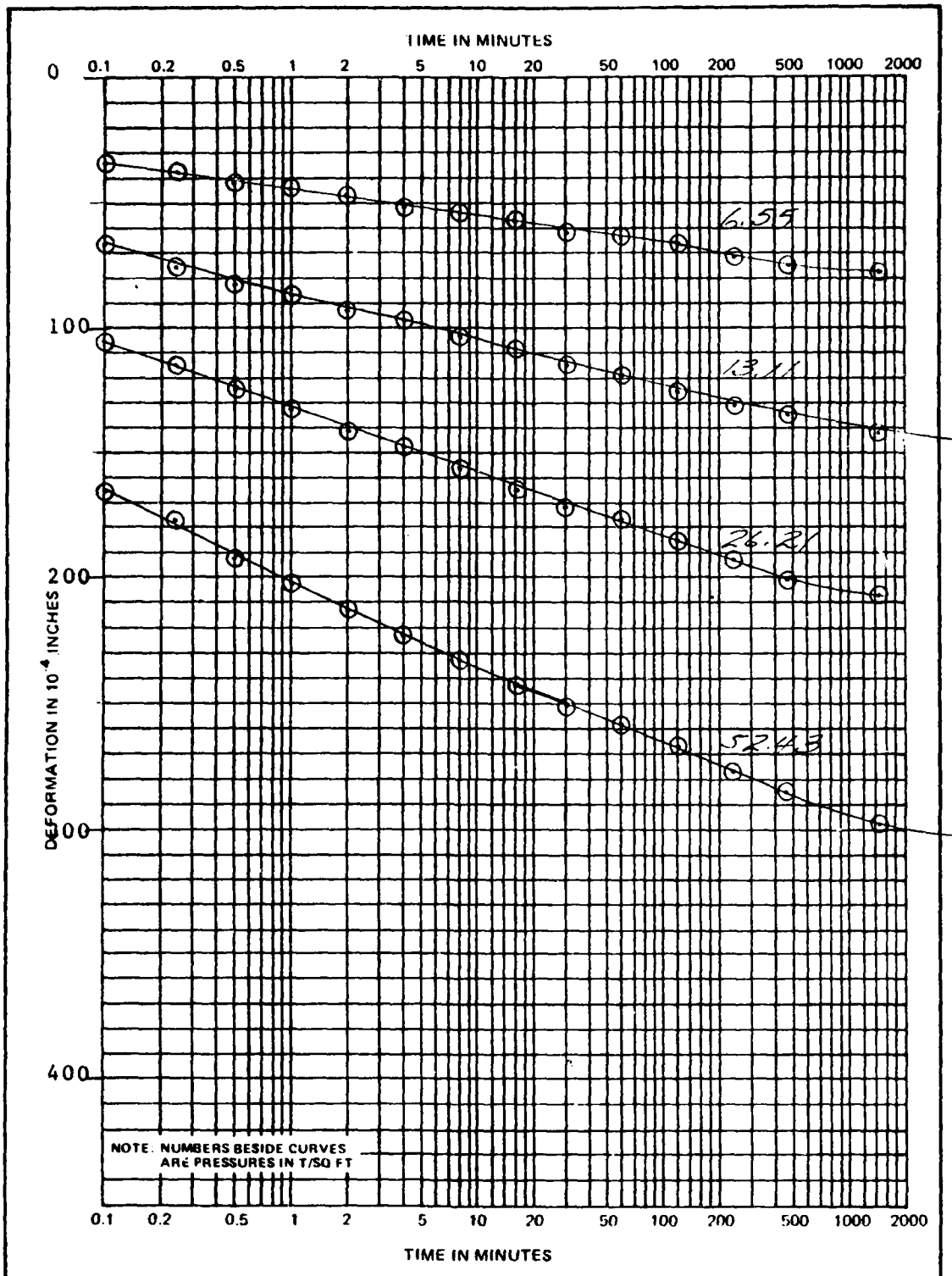
PLATE III, PHASE IV



PROJECT Seven Oaks Dam - Mill Creek Impervious Borrow			
AREA Div.No 100035 (Phase IV,Program 2)			
BORING NO CBH 86-29	SAMPLE NO Blend 7A	DEPTH EL 0.0'-10.0'	DATE October1987

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST–TIME CURVES** (TRANSLUCENT)

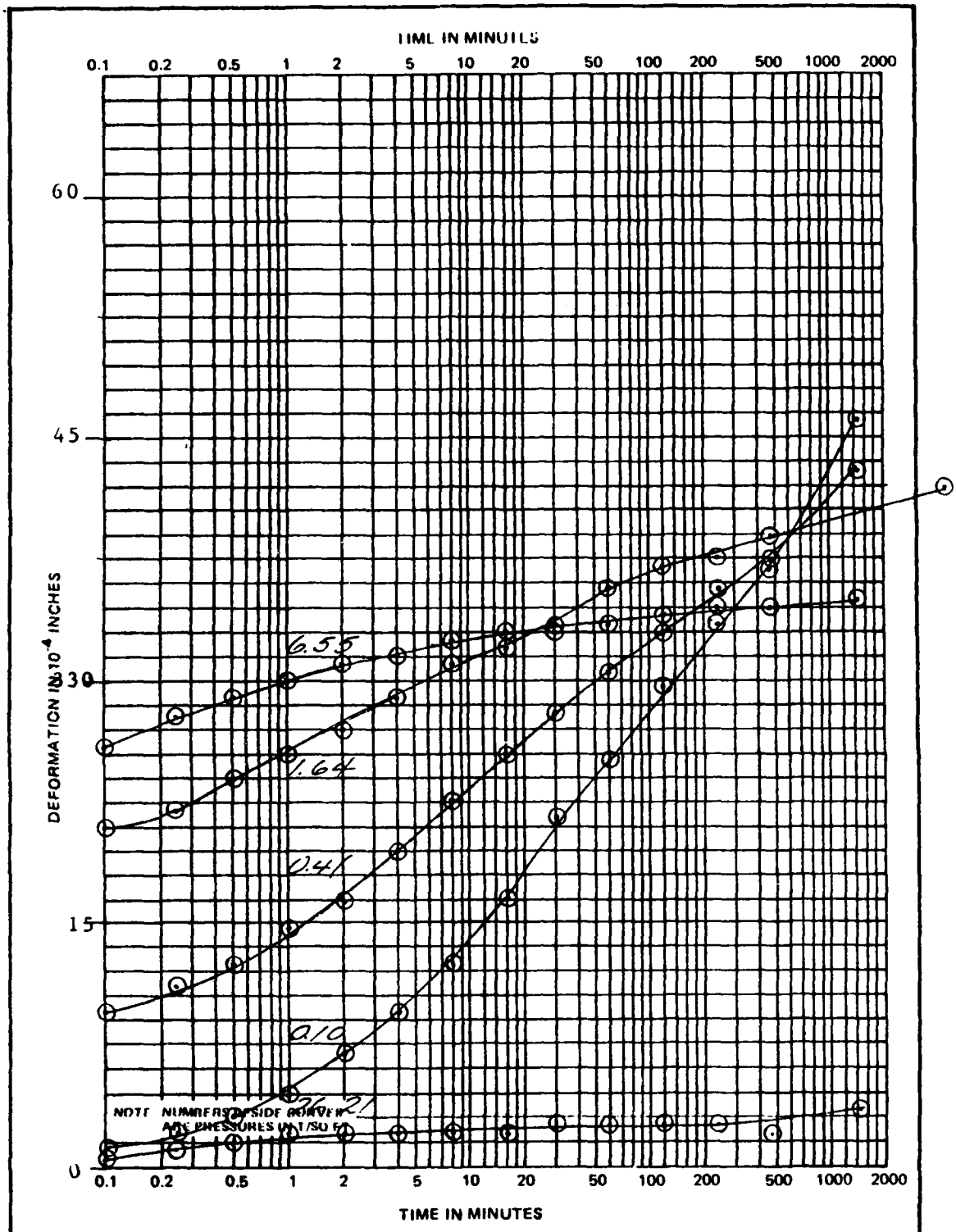
PLATE 12, PHASE IV



PROJECT Seven Oaks Dam - Mill Creek Impervious Borrow			
AREA Div.No 100035 (Phase IV, Program 2)			
BORING NO CBH 86-29	SAMPLE NO Blend 7A	DEPTH EL 0.0±10.0'	DATE October 1987

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST–TIME CURVES** (TRANSLUCENT)

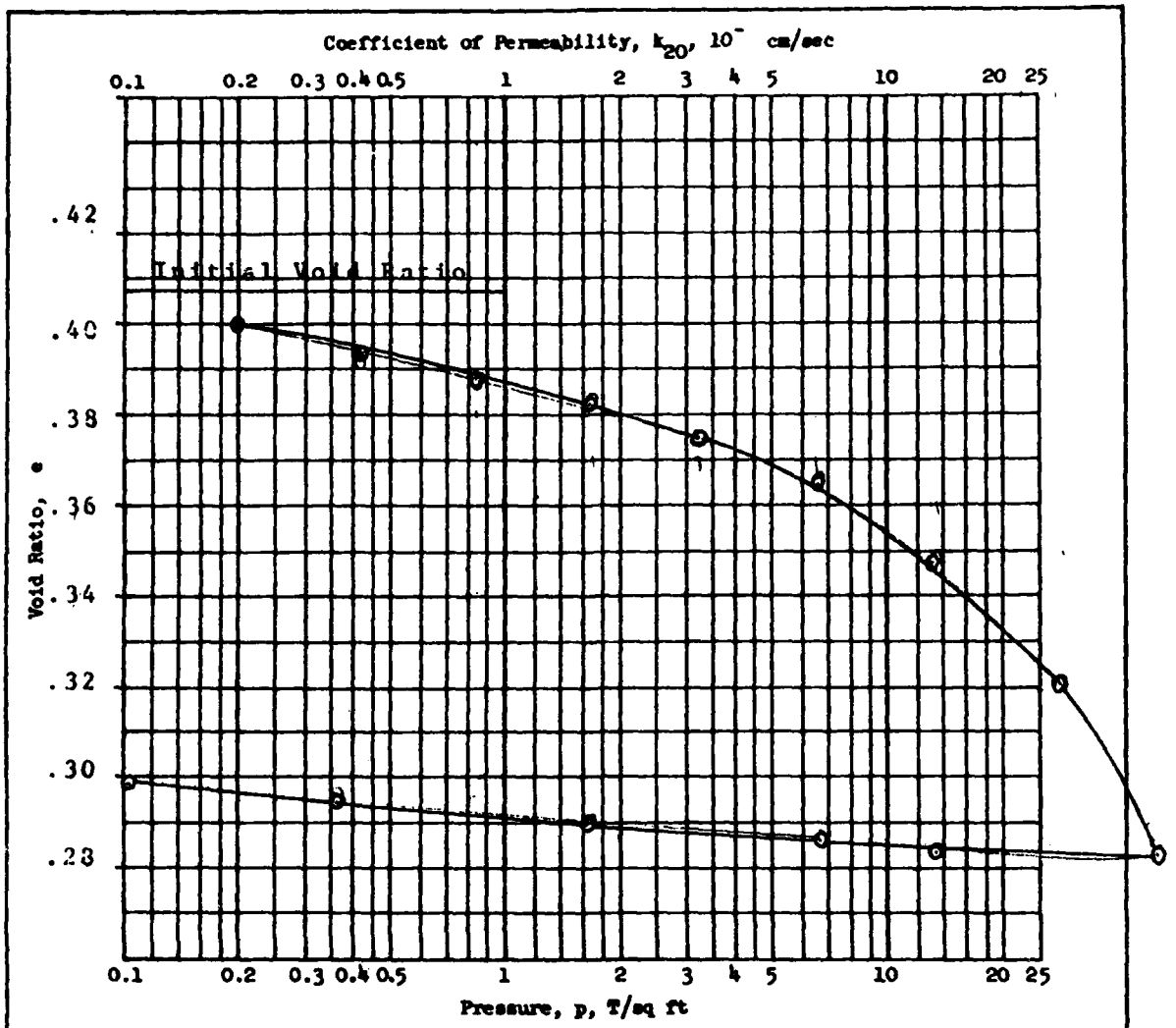
PLATE 15, PHASE IV



PROJECT	Seven Oaks Dam - Mill Creek Impervious Borrow			
AREA	Div.No 100035 (Phase IV, Program 2)			
BORING NO.	CBH 86-29	SAMPLE NO	Blend 7A	DEPTH EL 0.0'-10.0'
DATE	October 1987			

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST--TIME CURVES** (TRANSLUCENT)

PLATE 11A, PHASE IV

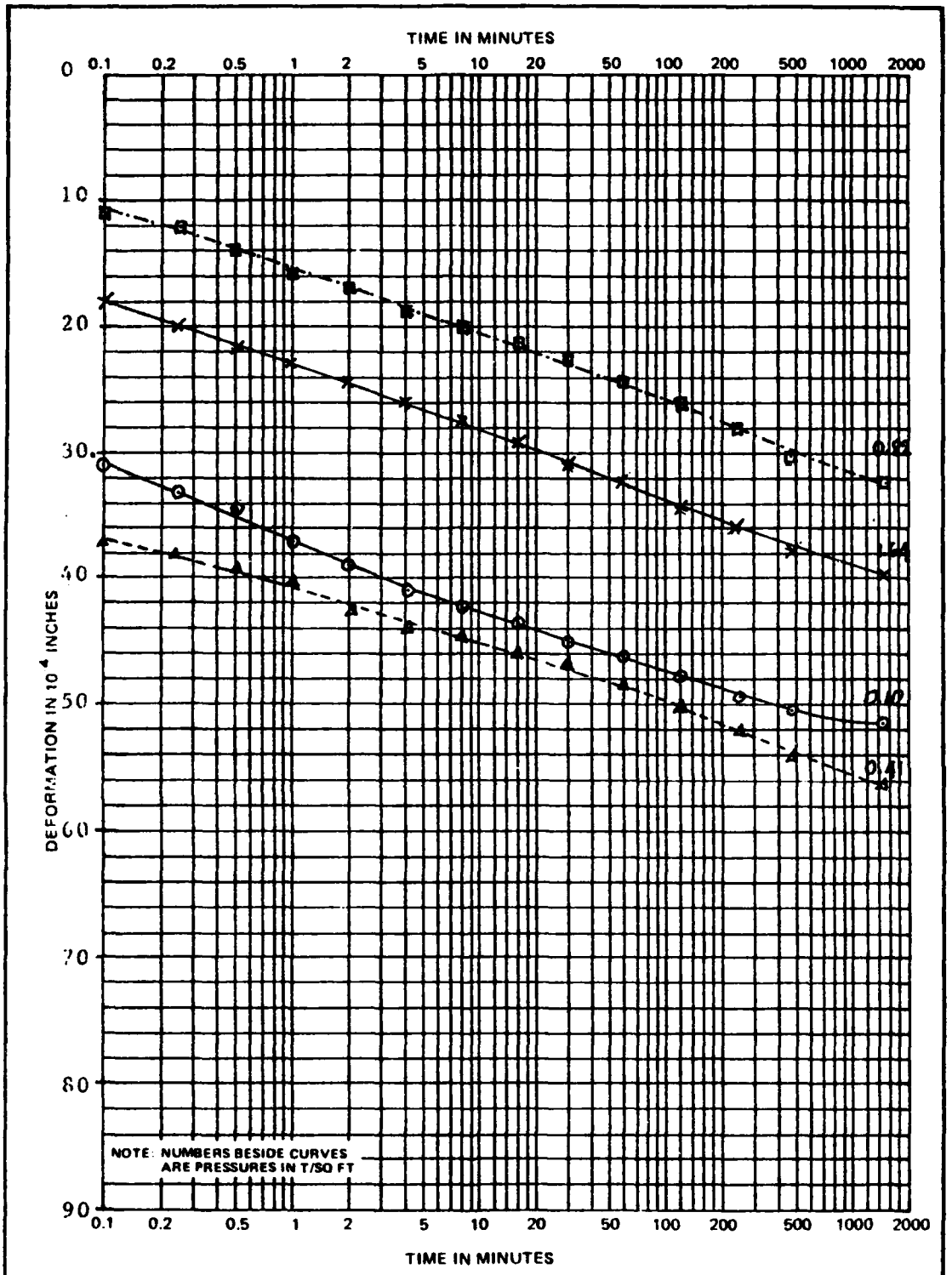


Type of Specimen Remolded		Before Test		After Test	
Diam 4.4 in.	Ht 1.0 in.	Water Content, w_o	12.9 %	w_f	11.4 %
Overburden Pressure, Po T/sq ft		Void Ratio, e_o	0.407	e_f	0.302
Preconsol. Pressure, Pc T/sq ft		Saturation, S_o	85 %	S_f	100 %
Compression Index, C_c		Dry Density, γ_d	118.8 lb/ft^3		
Classification Silty Sand (SM)		k_{20} at e_o =	$\times 10^{-}$ cm/sec		
LL	G_s 2.68	Project Seven Oaks dam			
PL	D_{10}	Mill Creek Impervious Borrow			
Remarks Div.No. 100050		Area (Phase IV, Program 2)			
Remolded to 98% maximum		Boring No. CBH 86-36		Sample No. Blend 6A	
density at optimum +2%.		Depth El 7.0'-12.0'		Date Oct. 1987	

CONSOLIDATION TEST REPORT

ENG FORM 1 MAY 68 2090 PREVIOUS EDITIONS ARE OBSOLETE.

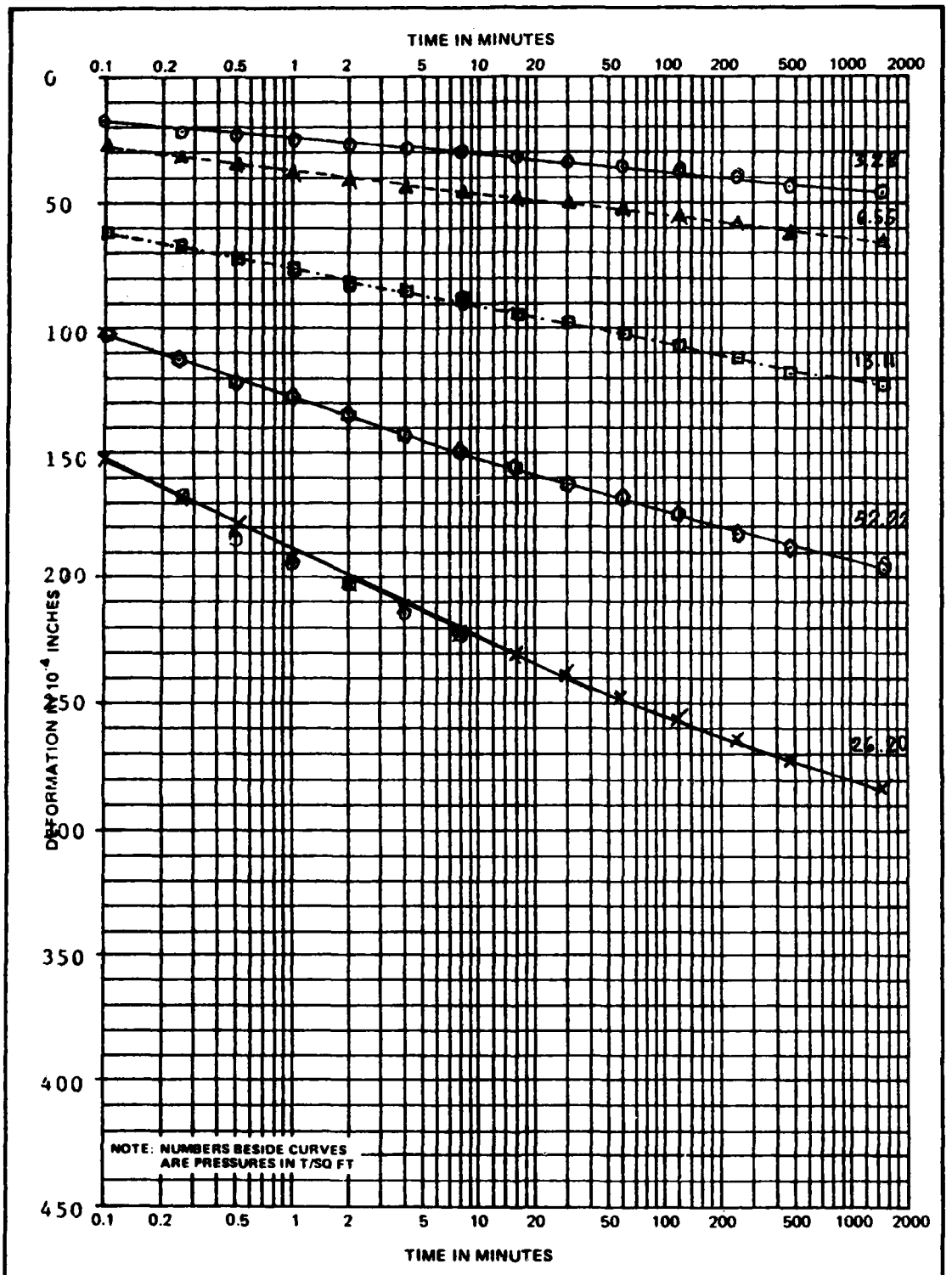
PLATE 15, PHASE IV



PROJECT	Seven Oaks Dam - Mill Creek Impervious Borrow			
AREA	Div. No. 100050 (Phase IV, Program 2)			
BORING NO. CRH 86-38	SAMPLE NO. Blend 6A	DEPTH EL 7.0'-12.0'	DATE Oct. 1987	

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST–TIME CURVES** (TRANSLUCENT)

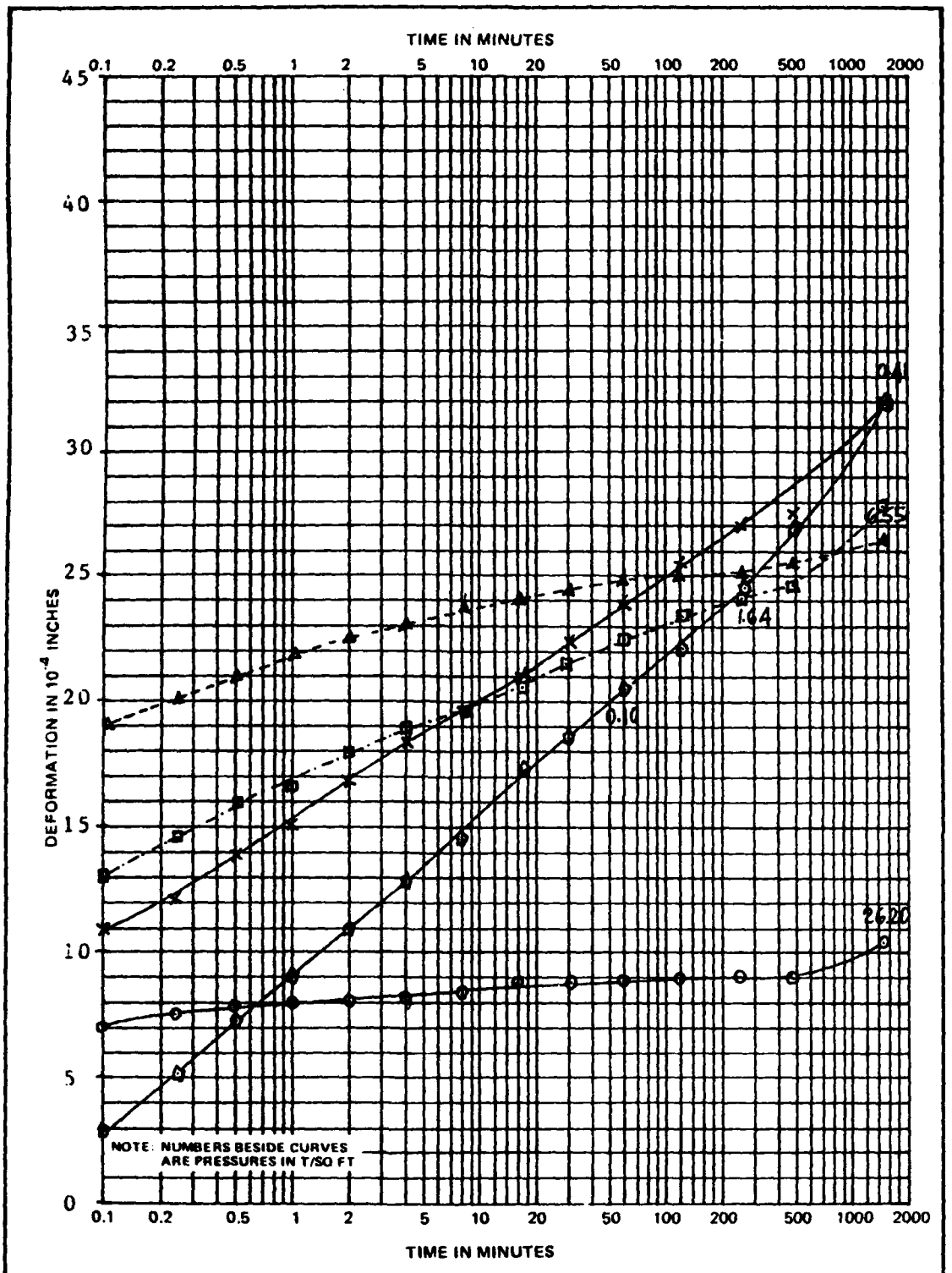
PLATE 116, PHASE IV



PROJECT Seven Oaks Dam - Mill Creek Impervious Borrow			
AREA Div.No. 100050 (Phase IV, Program 2)			
BORING NO. CBH 86-38	SAMPLE NO Blend 6A	DEPTH EL 7.0'-12.0'	DATE Oct. 1987

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST–TIME CURVES** (TRANSLUCENT)

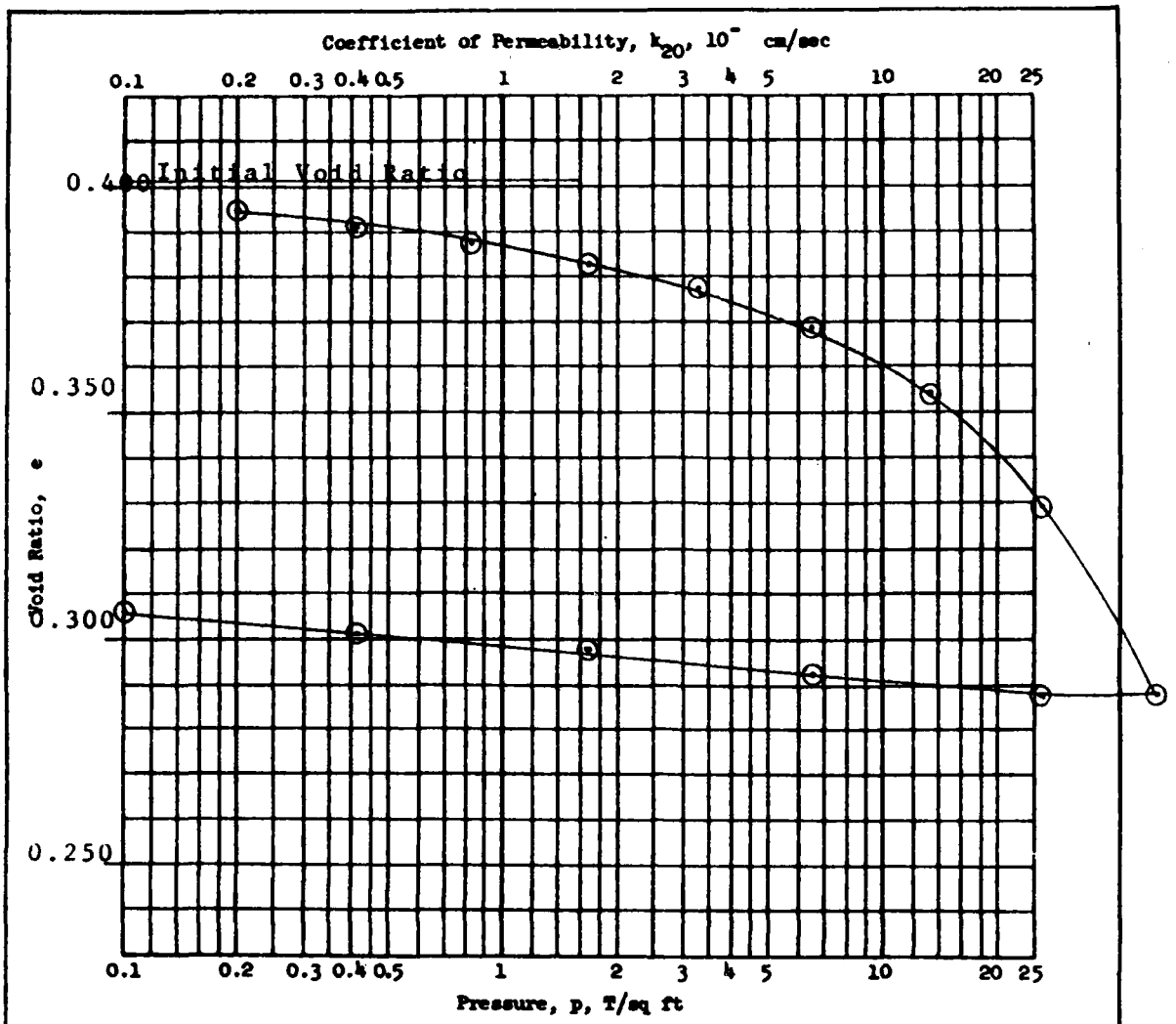
PLATE 117, PHASE IV



PROJECT	Seven Oaks Dam - Mill Creek Impervious Borrow			
AREA	Div.No. 100050 (Phase IV, Program 2)			
BORING NO.	CBH 86-36	SAMPLE NO.	Blend 5A	DEPTH EL 7.0'-12.0'
DATE	Oct. 1987			

ENG FORM 1 MAY 83 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST–TIME CURVES** (TRANSLUCENT)

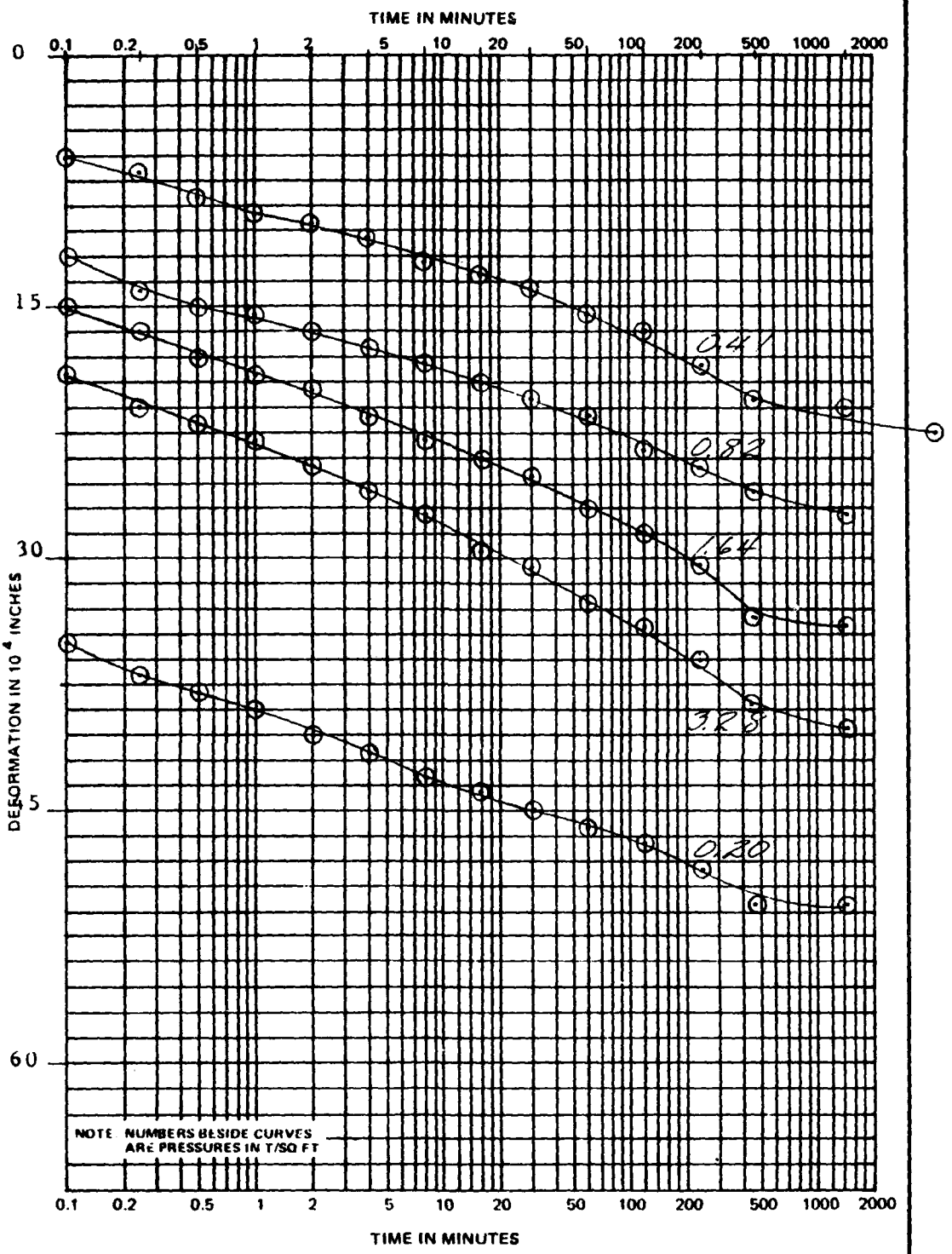
PLATE IIB , PHASE IV



Type of Specimen Remolded		Before Test		After Test	
Diam 4.4 in.	Ht 1.0 in.	Water Content, w_o	10.9 %	w_f	11.0 %
Overburden Pressure, p_o	T/sq ft	Void Ratio, e_o	0.401	e_f	0.308
Preconsol. Pressure, p_c	T/sq ft	Saturation, S_o	73 %	S_f	96 %
Compression Index, C_c		Dry Density, γ_d	118.9 lb/ft^3		
Classification Clayey Sand (SC)		k_{20} at e_o =	x 10^{-} cm/sec		
LL	G_s 2.68	Project Seven Oaks Dam			
PL	D_{10}	Mill Creek Impervious Borrow			
Remarks Div. No 100050		Area (Phase IV, Program 2)			
Remolded to 98% maximum		Boring No. CBT 86-38	Sample No Blend 6A		
density at optimum.		Depth El 5.0'-11.0'	Date October 1987		

CONSOLIDATION TEST REPORT

ENG FORM 1 MAY 69 2090 PREVIOUS EDITIONS ARE OBSOLETE.



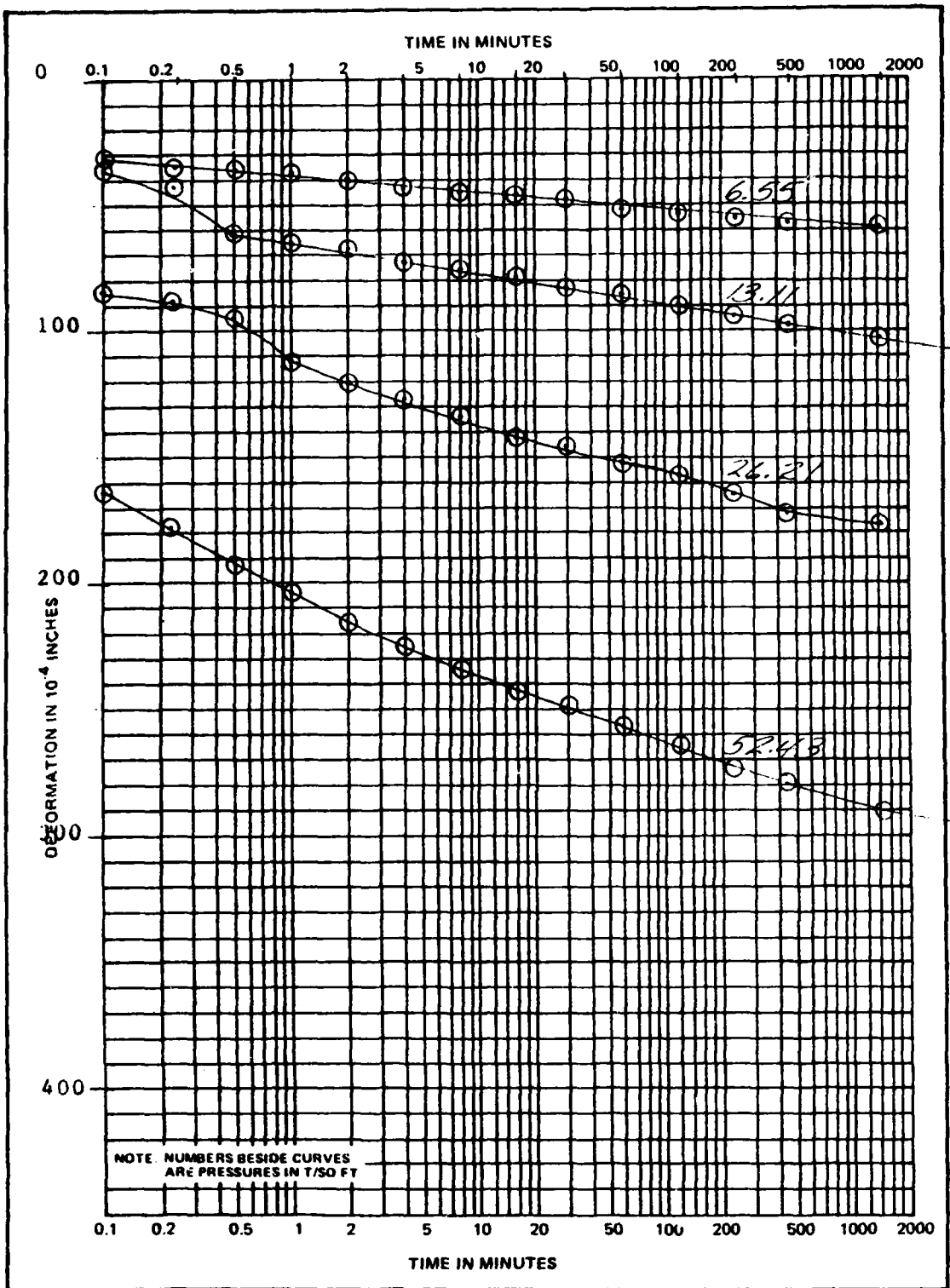
PROJECT Seven Oaks Dam - Mill Creek Impervious Borrow

AREA Div.No 100050 (Phase IV, Program 2)

BORING NO. CBT-86-38 SAMPLE NO Blend A DEPTH EL 5.0'-11.0' DATE October 1987

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST--TIME CURVES** (TRANSLUCENT)

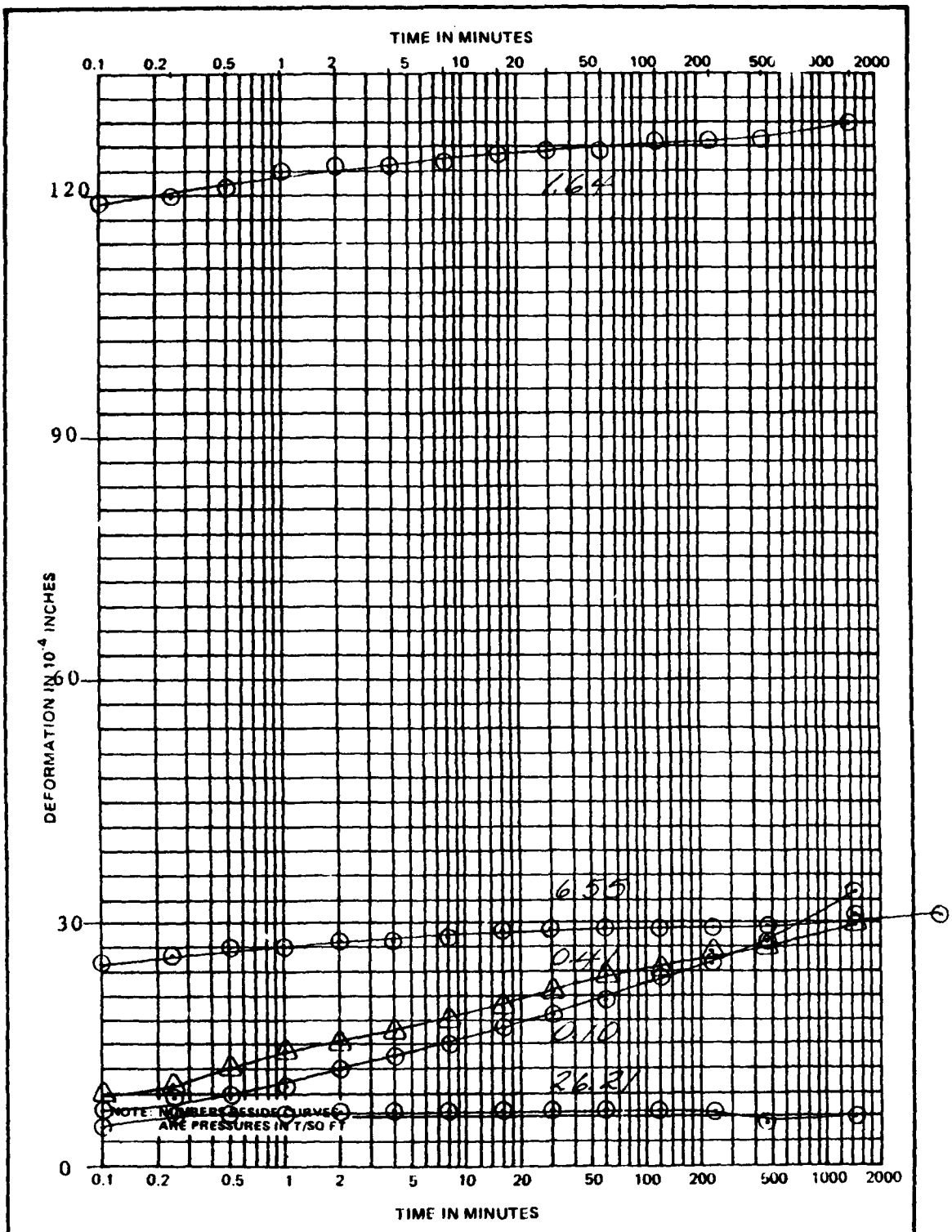
PLATE 20 PHASE IV



PROJECT Seven Oaks Dam - Mill Creek Impervious Borrow			
AREA Div.No 100050 (Phase IV, Program 2)			
BORING NO. CBT 86-38	SAMPLE NO Blend6 .A	DEPTH EL 5.0'-11.0'	DATE October 1987

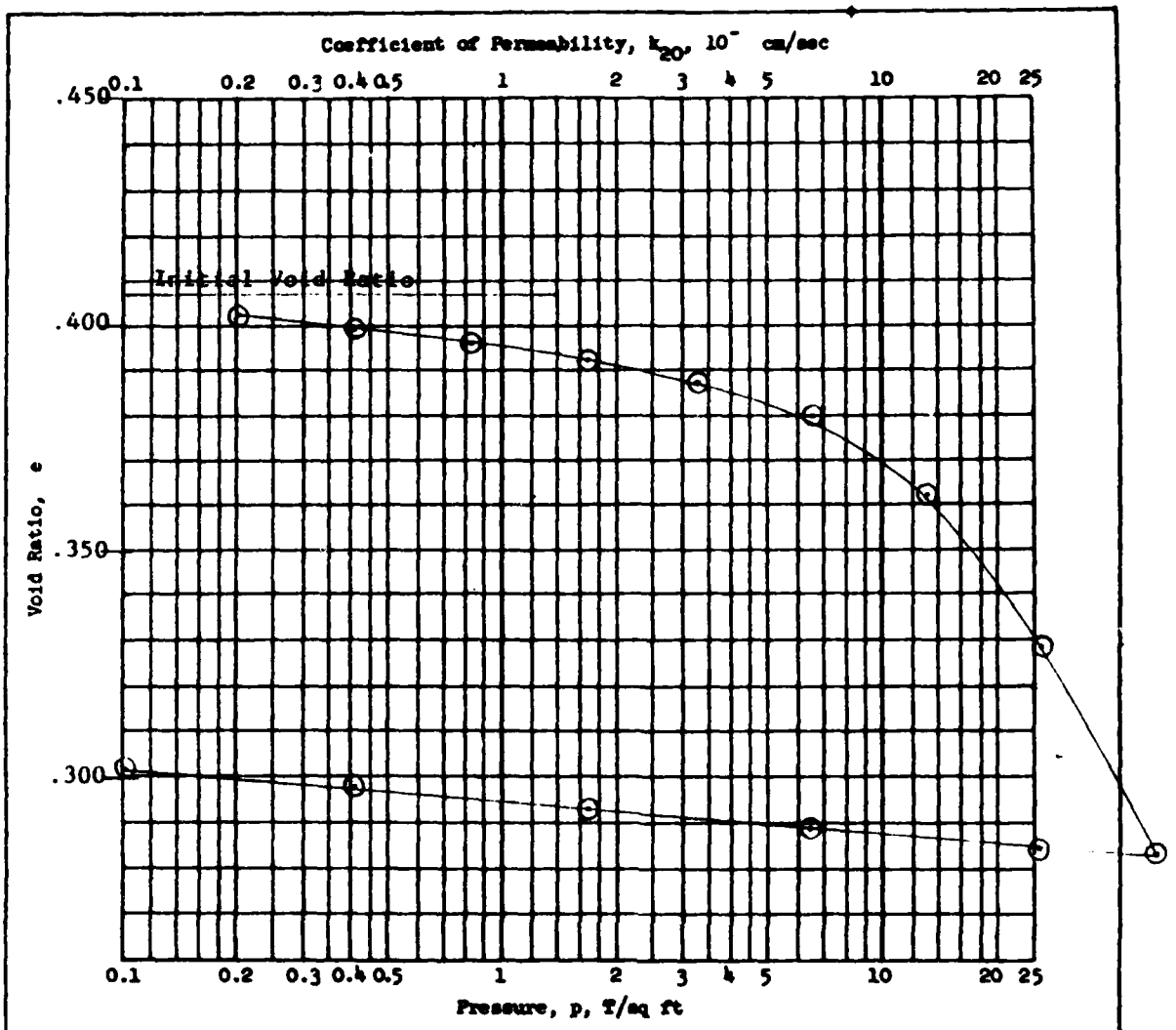
ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST–TIME CURVES** (TRANSLUCENT)

PLATE (21), PHASE IV



PROJECT	Seven Oaks Dam - Mill Creek Impervious Borrow			
AREA	Div.No 100050 (Phase IV, Program 2)			
BORING NO.	CBT 86-38	SAMPLE NO.	Blend 6A	DEPTH EL 5.0'-11.0'
DATE	October 1987			

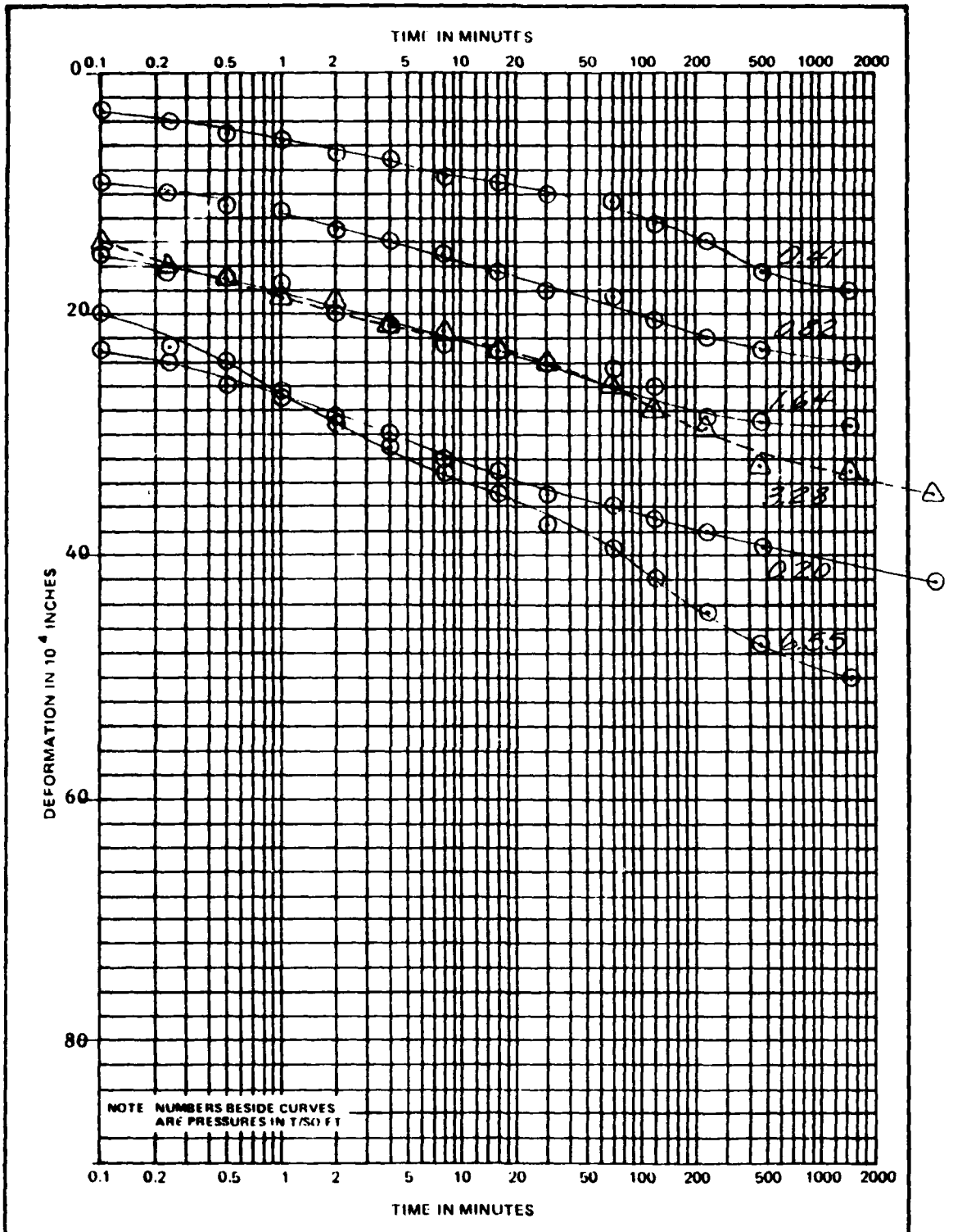
ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST–TIME CURVES** (TRANSLUCENT)



Type of Specimen Remolded		Before Test		After Test	
Diam 4.4 in.	Ht 1.0 in.	Water Content, w_o	9.0 %	w_f	10.6 %
Overburden Pressure, p_o T/sq ft		Void Ratio, e_o	.408	e_f	.304
Preconsol. Pressure, p_c T/sq ft		Saturation, S_o	59.0 %	S_f	94.0 %
Compression Index, C_c		Dry Density, γ_d	118.8 lb/ft^3		
Classification Silty Sand (SM)		k_{20} at e_o =	x 10^{-} cm/sec		
LL	G_s 2.68	Project SEVEN OAKS DAM			
PL	D_{10}	Mill Creek Impervious Borrow			
Remarks Div. No. 100050		Area Phase IV-Program 2			
98% maximum density at		Boring No. CBH 86-38	Sample No. Blend 6A		
optimum -2%		Depth El 7'-12'	Date Oct. 1987		

CONSOLIDATION TEST REPORT

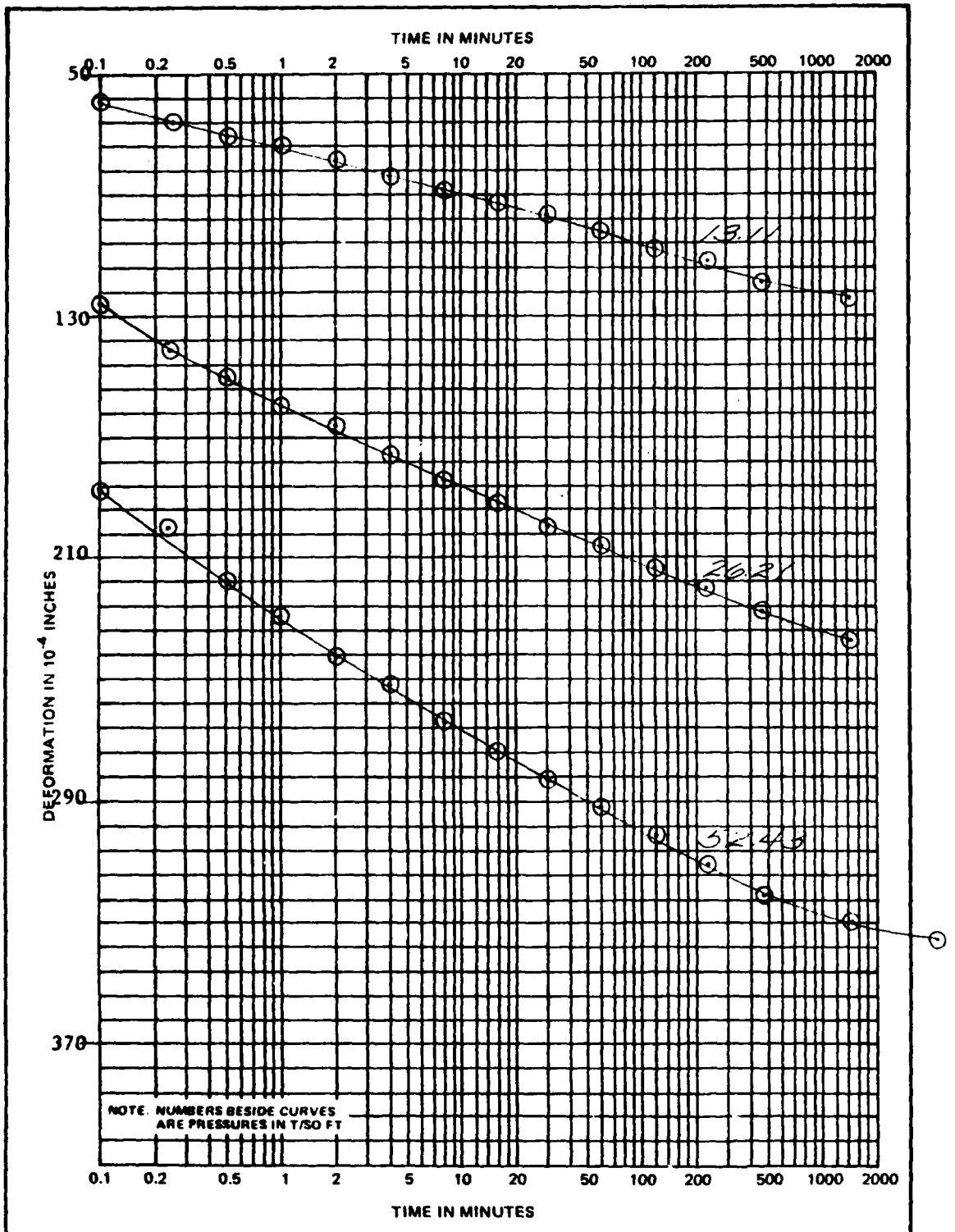
ENG FORM 1 MAY 68 2090 PREVIOUS EDITIONS ARE OBSOLETE.



PROJECT	SEVEN OAKS DAM - Div. No. 100050			
AREA	Mill Creek Impervious Borrow (Program 2, Phase IV)			
BORING NO.	CBH-86-38	SAMPLE NO Blend 6A	DEPTH EL 7'-12'	DATE Oct 1987

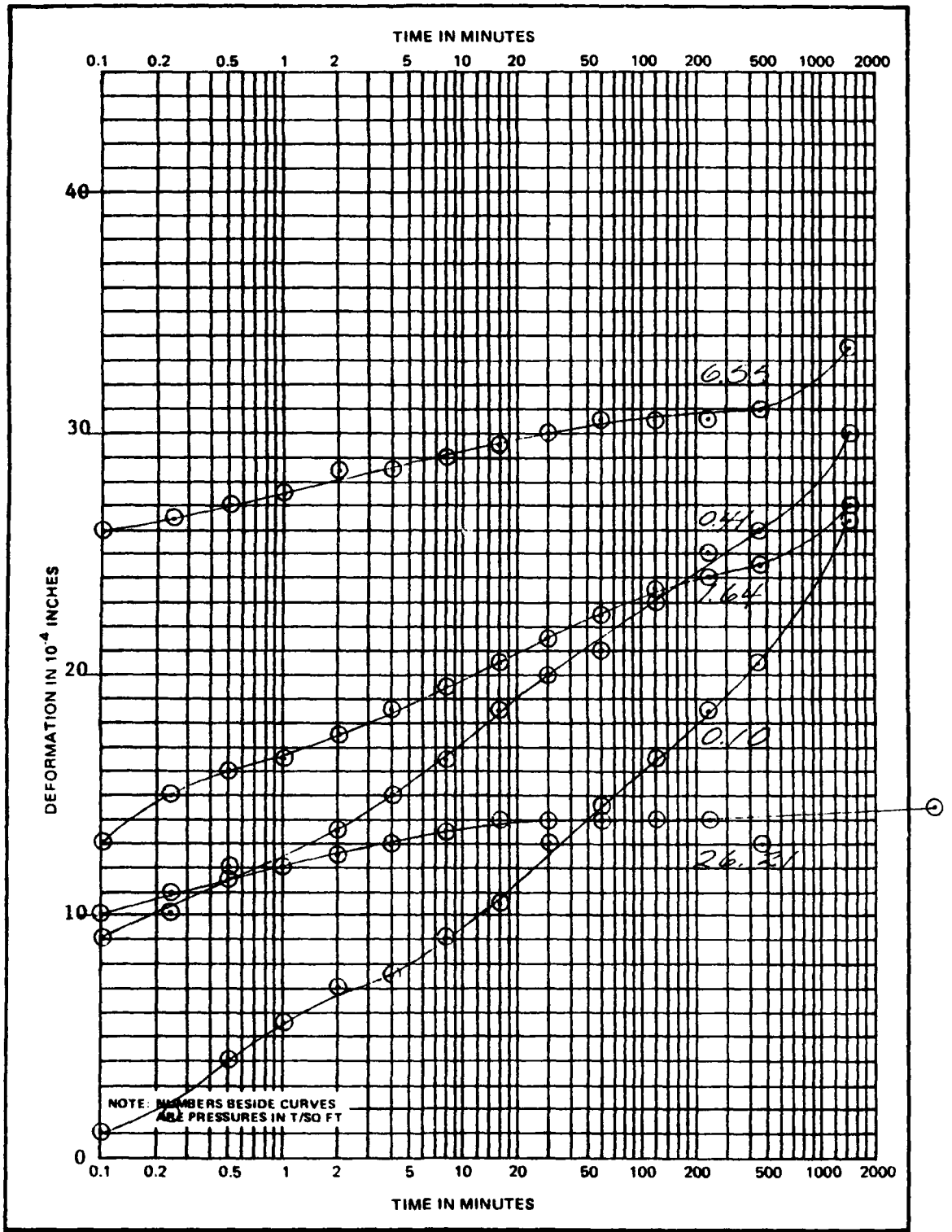
ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST–TIME CURVES** (TRANSLUCENT)

PLATE 24 PHASE IV



PROJECT	SEVEN OAKS DAM - Div. No. 100050			
AREA	Mill Creek Impervious Borrow (Program 2, Phase IV)			
BORING NO. CBH-86-38	SAMPLE NO Blend 6A	DEPTH EL 7'-12'	DATE Oct 1987	
ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE	**CONSOLIDATION TEST–TIME CURVES** (TRANSLUCENT)			

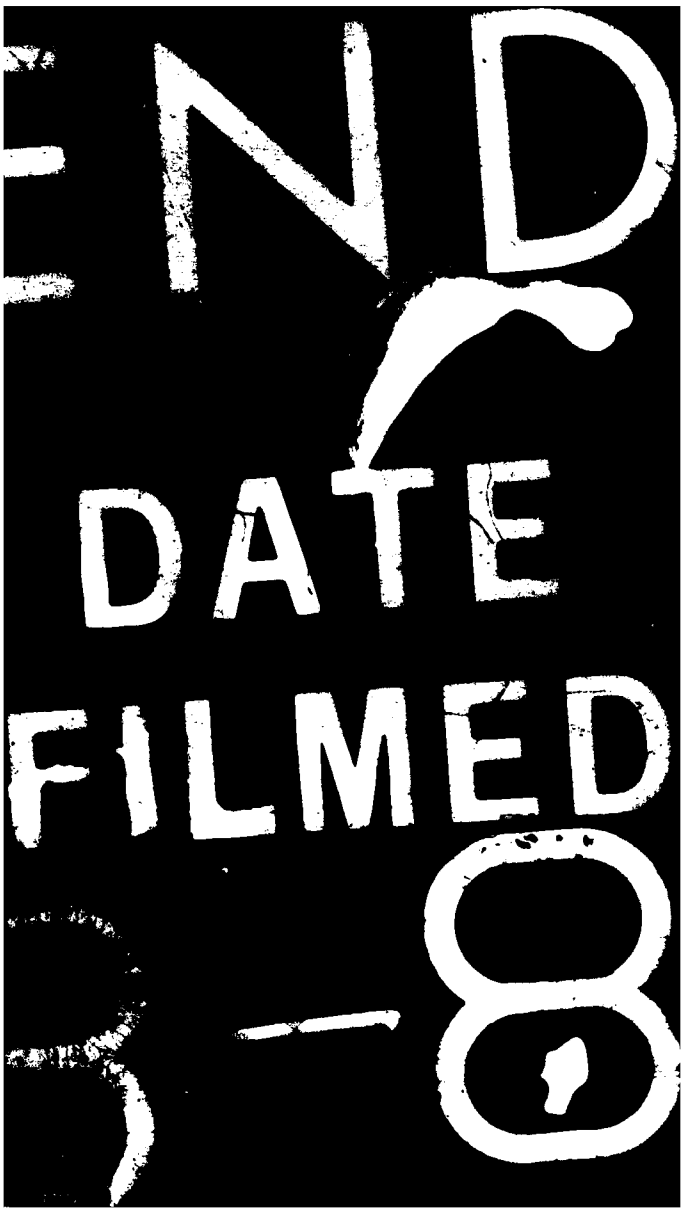
PLATE 25, PHASE IV



PROJECT	SEVEN OAKS DAM - Div. No. 100050			
AREA	Mill Creek Impervious Borrow (Program 2, Phase IV)			
BORING NO. CBH-86-38	SAMPLE NO. Blend 6A	DEPTH 7'-12' EL	DATE Oct 1987	

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE **CONSOLIDATION TEST—TIME CURVES** (TRANSLUCENT)

PLATE 26 PHASE IV

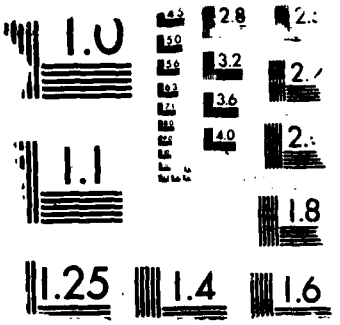

END
DATE
FILMED
-8



AD-A204 540 SANTA ANA RIVER DESIGN MEMORANDUM NUMBER 1 PHASE 2 GDM 1/1
ON THE SANTA ANA R..(U) ARMY ENGINEER DISTRICT LOS
ANGELES CA AUG 88

UNCLASSIFIED F/G 13/2 NL




1.0
1.1
1.25
1.4
1.6
1.8
2.8
3.2
3.6
4.0

SUPPLEMENTARY

INFORMATION

Errata

AD-A204 540

DESIGN MEMORANDUM NO. 1
PHASE II GDM ON THE SANTA ANA
RIVER MAINSTEM, INCLUDING SANTIAGO CREEK
DATED: AUGUST 1988

On November 15, 1988, the U.S. Army Corps of Engineers released, for agency and public review, the Congressionally authorized final General Design Memorandum (GDM) for the Santa Ana River Mainstem Project, including the Main Report and Supplemental Environmental Impact Statement, and accompanying volumes and technical appendixes. The Corps mailed copies of the final Phase II GDM to selected Federal, State, and local governmental agencies; elected officials within the project area; flood control districts; interest groups; and public libraries. Review of the final Phase II GDM by the agencies and the public generated comments which, in general, focused on the following concerns:

(a) Recreation trails along the Santa Ana River; (b) Aquatic habitat at Seven Oaks Dam; (c) Lower Santa Ana River sediment transport and potential beach degradation; (d) Esthetics and construction phasing for the Santiago Creek channel improvements; and (e) Flood threat and associated impacts to the Corona Airport within the Prado Dam. Inclosure 1 presents a synopsis of the U.S. Army Corps Engineers response to these concerns.

Following publication of the final Phase II GDM, several inconsistences were noted that require clarification of certain statements, and correction of typographical errors. The errata sheets (Inclosure 2) provide the revisions to be incorporated in the final Phase II GDM.

For additional information you may direct your inquiries to the following:

U.S. Army Corps of Engineers
P.O. Box 2711, L. A., CA 90053-2325
Attn: Mr. Dionicio Gonzales
Tel No. (213) 894-2713

Brian M. Moore

Encls

Tadahiko Ono
for Colonel, Corps of Engineers
District Engineer

U.S. ARMY CORPS OF ENGINEERS
RESPONSES TO AGENCY AND PUBLIC REVIEW COMMENTS
ON
DESIGN MEMORANDUM NO. 1
PHASE II GENERAL DESIGN MEMORANDUM
ON THE SANTA ANA RIVER MAINSTEM, INCLUDING
SANTIAGO CREEK
DATED: AUGUST 1988

RECREATION TRAILS ALONG THE SANTA ANA RIVER

ISSUE - The Main Report and Supplemental Environmental Impact Statement indicated that the equestrian trails in several locations along the Santa Ana River would be a continuous paved surface. Comment was made that this would constitute an unacceptable conversion of use since the existing trails are unpaved.

RESPONSE - The Corps and Orange County, one of the sponsors, will be developing several alternative solutions in coordination with the National Park Service to resolve this issue. One solution would be to locate the trail along the toe of the levee, while a more promising one may be to use excess spoil material to widen the top of the levee within the existing rights-of-way. We anticipate that this issue can be successfully resolved.

AQUATIC HABITAT AT SEVEN OAKS DAM

ISSUE - Concern was for the need for additional mitigation measures to compensate for impacts on aquatic habitats at the Seven Oaks damsite.

RESPONSE - The recommended mitigation plan to compensate for impacts resulting from the Seven Oaks Dam portion of the project was evaluated, and project related impacts and achievable mitigation goals were defined. The evaluation indicated that the mitigation plan for Seven Oaks Dam will meet 14.5% of the mitigation goal for aquatic habitat. Following coordination with the various resource agencies, no mitigation options were found which would achieve 100% mitigation under current Corps policy on mitigation. The Corps agrees that the aquatic habitat is impacted and has identified the magnitude of the impact according to NEPA requirements. The Corps has considered all practicable mitigation options in fulfilling its 404(b)(1) requirements. The project has been determined to not be contrary to the public interest even though 100% mitigation of impacts is not achieved.

LOWER SANTA ANA RIVER - SEDIMENT TRANSPORT & BEACH DEGRADATION

ISSUE - Concern was for impacts of the project on coastal beaches and that the project does not assure commitments to mitigate for these adverse impacts.

RESPONSE - The concern was based on the statement in the SEIS, page V-57, paragraph 5-192, which stated that there would be a reduction in sediment available for beach replenishment as a result of the project. Upon close scrutiny of the aforementioned paragraph we find that the statements contained therein are erroneous and was inadvertently included in the SEIS. Volume 3, Lower Santa Ana River, presents results of the sediment transport analysis which indicates that there would be a net increase of 11,000 cubic yards of sediment per year available for beach replenishment with the project in place. Accordingly, the aforementioned paragraph in the SEIS will be revised to read as follows:

"Under existing channel conditions, large floods will breach levees causing flood flows and sediment to exit and deposit onto the Santa Ana River Flood Plain. With the project channel improvements, large flood flows (up to 190 year frequency) will remain in the channel, thus causing any sediment that would have been deposited in the floodplain to be deposited in the channel itself or conveyed to the ocean. With the Santa Ana River project in place, sand-sized sediment yield (average annual basis) is estimated to increase by 11,000 cubic yards."

The Corps has held several meetings with staff members of both the California Coastal Commission and the City of Newport Beach to resolve the issue of placement of compatible channel material on the beach. The discussions appear to be headed to a mutually acceptable agreement.

ESTHETICS AND CONSTRUCTION PHASING FOR THE SANTIAGO CREEK CHANNEL IMPROVEMENTS

ISSUE - The concern was raised regarding the channel design and construction phasing of Santiago Creek, and the associated esthetics impacts of the project.

RESPONSE - The design displayed in the report for the stabilization of Santiago Creek between the Santa Ana Freeway and the Santa Ana River reflects the minimum amount of construction required to reliably and economically protect the streambed and banks of the creek from erosion. This design was developed after carefully consideration of the desires of residents along the creek as expressed in numerous public involvement meetings, and as the result of detailed investigation of several alternatives. The Corps of Engineers cannot support any lesser level of improvement as being sufficiently reliable. If this reach of channel is not stabilized to the minimum level shown in the report, significant erosion of the stream banks with potentially disastrous damage to property immediately adjacent to the creek on both sides would occur during controlled design flood releases from the detention basin. If the reach of the Santiago Creek from the Santa Ana Freeway to the Santa Ana River is not sufficiently stabilized, the flood control project cannot be safely operated as designed. In regards to the construction phasing for Santiago Creek it is not advisable to construct the upstream flood control improvement <u>prior</u> to commencing

any construction downstream of the freeway because of the need to have the lower channel in place to operate the detention pits.

FLOOD THREAT TO THE CORONA AIRPORT WITHIN PRADO BASIN

ISSUE - Concerns were raised about the potential flood threat at the Corona Airport as a result of the Corps recommended modifications to Prado Dam.

RESPONSE - The Corps studies indicate that the recommended Prado modifications will enable us to make larger releases from Prado Dam, thus allowing faster drawdown of the flood control pool. Consequently, within the period of the current airport lease, the frequency and duration of flooding at the airport will be reduced with the recommended modification of Prado Dam. Should interests at the airport feel that a levee is imperative to protect the airport from frequent flooding while allowing impoundment during major storm events, they would need to identify a local financial sponsor to bear the full costs for the levees and for the costs for mitigative measures resulting from the construction of the levees. These costs are entirely non-Federal expenses. It is noted that the Corona Airport is located on lands owned by the Federal Government for the purpose of flood control and all investments in this location were made with the full knowledge of the flood threat. As our recommended modifications will not result in more frequent or longer durations of flooding, the Corps did not include flood protection features at this location.

AD-A204 540

VOLUME 1

SEVEN OAKS DAM

ERRATA

Page XVIII - 1 para. 18-02, insert "Regardless of who performs the work, operations and maintenance of Seven Oaks Dam is the responsibility of the local sponsors" to the end of the paragraph.